# JOURNAL

OF THE

# AMERICAN WATER WORKS ASSOCIATION

VOL. 23 DECEMBER, 1931 No. 12

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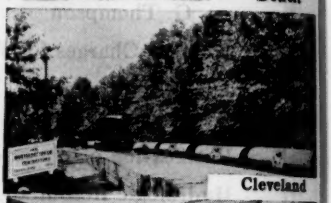


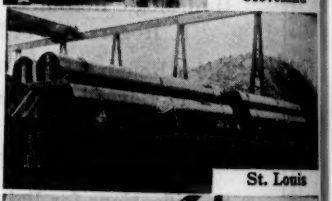

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# JOURNAL

OF THE

# AMERICAN WATER WORKS ASSOCIATION

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*Discussion of all papers is invited*

Vol. 23 DECEMBER, 1931 No. 12


## A REPORT ON FAILURES OF WATER MAINS[1]

At a meeting of the Council on March 1, 1929, a subcommittee was appointed to enquire into the question of the failures of water mains. The terms of reference to the committee were as follows:

(*a*) To ascertain whether there has been in recent times, and particularly in the past season, a general abnormal number of bursts.

(*b*) If so, to investigate their causes.

(*c*) To advise on measures necessary to prevent such failures.

(*d*) To consider whether joint action with gas authorities and other interested subsurface users is advisable.

At its first meeting the committee decided that the best method of investigating the questions referred to them was by direct enquiry to water authorities throughout the country, and accordingly a questionnaire and tabular forms were prepared and sent to all members of the Institution in charge of works in this country and to some abroad.

Briefly, the main points concerning which information was requested were as follows:

1. Particulars of the number of bursts on pipes of all sizes in the last five years, with corresponding figures for a similar period of five years with an interval of ten years between the two periods.

[1] Issued by the Institution of Water Engineers, London, England, in June, 1931.

TABLE 1

*Lengths of mains in miles in the year ended March 31, 1929*

| SIZE OF MAIN | 1 ARBROATH | 2 BARNSLEY | 3 BIRKENHEAD | 4 BRISTOL | 5 DURHAM COUNTY | 6 GLASGOW | 7 GREENOCK | 8 HALIFAX | 9 HULL | 10 LEEDS | 11 LEICESTER | 12 OLDHAM | 13 SCARBOROUGH | 14 SWANSEA | 15 WEST CHESHIRE | 15 AUTHORITIES Total | 15 AUTHORITIES Percent of total | 32 AUTHORITIES Total | 32 AUTHORITIES Percent of total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| *inches* | | | | | | | | | | | | | | | | | | | |
| 2 | — | — | 2 | — | — | — | — | — | — | — | — | — | — | — | — | 2 | 0.05 | 5.8 | 0.03 |
| 2½ | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | 62.2 | 0.3 |
| 3 | 15 | 30 | 31 | 215 | 345 | 27 | 2.5 | 75 | 99.5 | 204 | 184 | 113 | 19 | 25.5 | 48 | 1,433.5 | 29.6 | 4,772.1 | 24.6 |
| 4 | 5 | 24 | 52 | 109 | 112 | 416 | 31 | 72.5 | 132 | 242 | 129 | 51 | 11.5 | 102 | 32 | 1,521 | 31.4 | 7,197.6 | 37.1 |
| 5 | 3 | 9 | — | 39.5 | 32 | 5 | 0.5 | 2.5 | 2.5 | 13 | 3.5 | 1 | 5.5 | 3.5 | 4.5 | 125 | 2.6 | 981.3 | 5.1 |
| 6 | — | 12 | 39 | 15.5 | 34 | 125 | 14 | 36.5 | 49 | 69 | 57 | 37 | 4 | 32 | 39 | 563 | 11.6 | 1,785.1 | 9.2 |
| 7 | — | 1.5 | — | 26.5 | 8 | 3 | — | 0.5 | — | 8 | 4.5 | — | 2.5 | 2 | — | 56.5 | 1.2 | 607.0 | 3.1 |
| 8 | 1 | 4 | 2.5 | 21 | 12 | 8 | 6 | 16.5 | — | 18 | — | 12 | 2.5 | 0.5 | — | 104 | 2.2 | 315.1 | 1.6 |
| 9 | 1 | 0.5 | 6 | 12 | 43 | 98 | 1 | 2.5 | 17.5 | 13 | 13.5 | 0.5 | 2 | 18.5 | 10 | 239 | 4.9 | 605.4 | 3.1 |
| 10 | — | 2.5 | 3 | 11 | 19 | 3 | 1.5 | 7 | 18 | 18 | 7 | 11 | 1 | 4.5 | 5 | 111.5 | 2.3 | 387.8 | 2.0 |
| 12 | 1 | 7.5 | 7.5 | 9 | 33 | 45 | — | 6.5 | 14 | 12 | 16 | 8 | 2.5 | 10 | 11 | 183 | 3.8 | 835.9 | 4.3 |
| 13 | — | — | — | — | — | — | 7* | — | — | — | — | — | 6 | — | 1 | 14 | 0.3 | 14.0 | 0.06 |
| 14 | — | 8 | 3 | — | — | — | — | 3.5 | 2 | — | — | — | 0.5 | 9 | 7.5 | 33.5 | 0.7 | 66.3 | 0.3 |
| 15 | — | — | 5 | 2 | 12 | 20 | — | — | — | 9 | 1 | — | 1.5 | — | — | 50.5 | 1.1 | 212.0 | 1.1 |
| 16 | — | — | — | — | — | — | 2.5† | — | — | — | — | — | — | — | 0.5 | 3 | 0.05 | 22.8 | 0.1 |
| 17 | — | 0.5 | — | — | — | — | — | — | — | — | — | — | — | 23 | — | 23.5 | 0.5 | 23.5 | 0.1 |
| 18 | — | — | 0.5 | 3.5 | 8 | 13 | 2 | 7 | — | 19 | 9.5 | 2 | — | 8 | — | 72.5 | 1.5 | 257.1 | 1.3 |
| 19 | — | — | 2.5 | — | — | — | — | — | — | — | — | — | — | — | — | 2.5 | 0.05 | 2.5 | 0.01 |
| 20 | — | 6.5 | 1.5 | — | — | — | — | — | — | 5 | 9 | — | — | — | — | 22 | 0.5 | 48.4 | 0.2 |

| | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 21 | — | — | 1.5 | 25.5 | — | 1 | — | — | — | — | 7.5 | — | — | — | — | 35.5 | 0.7 | 109.6 | 0.6 |
| 22 | — | 5.5 | — | — | — | — | 1‡ | — | — | — | — | — | — | — | — | 6.5 | 0.1 | 6.5 | 0.03 |
| 24 | — | — | — | 10.5 | 9 | 30 | — | — | — | 12 | 11 | 4 | — | 25 | — | 101.5 | 2.1 | 341.9 | 1.8 |
| 25 | — | — | 15 | — | — | — | — | — | — | — | — | — | — | — | — | 15 | 0.3 | 15 | 0.1 |
| 26 | — | — | — | — | — | — | — | — | — | — | — | 2 | — | — | — | 2 | 0.05 | 2.0 | 0.01 |
| 27 | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | — | 6.8 | 0.03 |
| 28 | — | — | 19 | — | — | — | — | — | — | — | — | — | — | — | — | 19 | 0.4 | 19.0 | 0.1 |
| 30 | — | — | 0.5 | 5 | 11 | 1 | — | — | — | 7 | — | — | — | — | — | 24.5 | 0.5 | 300.8 | 1.6 |
| 33 | — | — | 0.5 | — | — | — | — | — | — | — | — | — | — | — | — | 0.5 | 0.05 | 6.3 | 0.03 |
| 36 | — | — | — | — | — | 69 | — | — | — | — | — | — | — | — | — | 69 | 1.4 | 227.1 | 1.2 |
| Above 36 | — | — | — | — | — | — | — | — | — | 4 | — | — | — | — | — | 4 | 0.05 | 170.1 | 0.9 |
| Totals..... | 26 | 111.5 | 192 | 505 | 678 | 864 | 69 | 230 | 334.5 | 653 | 452.5 | 241.5 | 58.5 | 263.5 | 158.5 | 4,837.5 | 100.0 | 19,407.0 | 100.0 |
| Percent of totals.... | 0.5 | 2.3 | 4.0 | 10.4 | 14.0 | 17.8 | 1.4 | 4.8 | 6.9 | 13.5 | 9.4 | 5.0 | 1.2 | 5.5 | 3.3 | — | 100.0 | — | — |

*Note.* Greenock figures.

* The length given of 7 miles is for 12- and 13-inch pipes—respective lengths unknown.

† The length given of 2.5 miles is for 14- and 16-inch pipes—respective lengths unknown.

‡ The length given of 1 mile is for 21- and 22-inch pipes—respective lengths unknown.

2. The relative number of bursts in frost and non-frost periods.
3. Whether mains on which bursts occurred were carrying gravitation or well water.
4. The kind of joint used, and, if of various types, the approximate proportion of each was asked for.
5. Whether mains generally were laid in the footway or carriageway.
6. Particulars as to depth of cover on mains.
7. Whether the pipes used were individually tested or not.

Replies were received from 35 authorities in all, but only 15 of these furnished complete records extending over the two periods necessary to enable the committee to deal with the first of the above questions. These authorities embraced such important places as Glasgow, Leeds, Leicester and Swansea, who gave complete information covering the periods 1909–1913 and 1924–1928.

Table 1 gives the lengths of mains of various sizes in commission in 1928 for each of these 15 authorities.

The figures in table 1 present several interesting features, one of the most striking being the variation in the proportion of small diameter mains to the total lengths for the different authorities.

The average percentage of 3-inch pipes in the total length of mains belonging to these 15 undertakings is rather higher than usual, but for the purpose of comparing the number of bursts occurring in the last few years with those of an earlier period, the authorities quoted are sufficiently representative as regards size and geographical position to enable a fair opinion to be deduced from their returns.

In order that more representative figures of the percentage lengths of mains of various sizes might be obtained, a statement similar to that relating to the 15 authorities as set out in table 1 was prepared, based on the returns of 32 (including the 15) authorities.

The total lengths and percentages of mains of different sizes for this larger number of undertakings are given in the last two columns of table 1.

Eight other authorities, including Liverpool, Manchester, Newcastle and Sunderland, supplied figures for two separate periods of five years, but in each of these cases, the interval between the two periods was less than ten years.

Ten other authorities, including the Metropolitan Water Board and Birmingham, supplied statistics for the last five years only, which, while not affording evidence as to the relative number of failures at the present time as compared with those in former years, have been useful in other directions.

## 1. FREQUENCY OF BURSTS IN RECENT AS COMPARED WITH FORMER YEARS

For the investigation of this part of the enquiry, the statements received from the 15 authorities set out in table 1 have been used.

TABLE 2

*Average number of bursts per mile*

(Based on returns from 15 authorities)

| SIZE | MEAN | | INCREASE | DECREASE |
|---|---|---|---|---|
| | First period (1909–1913) (calendar years) | Second period (1924–1928) (calendar years)* | | |
| *inches* | | | | |
| 3 | 0.79 | 0.79 | — | — |
| 4 | 0.56 | 0.53 | — | 0.03 |
| 5 | 0.52 | 0.55 | 0.03 | — |
| 6 | 0.37 | 0.33 | — | 0.04 |
| 7 | 0.35 | 0.30 | — | 0.05 |
| 8 | 0.30 | 0.34 | 0.04 | — |
| 9 | 0.51 | 0.49 | — | 0.02 |
| 10 | 0.28 | 0.28 | — | — |
| 12 | 0.30 | 0.32 | 0.02 | — |
| 14 | 0.14 | 0.23 | 0.09 | — |
| 15 | 0.34 | 0.47 | 0.13 | — |
| 18 | 0.26 | 0.19 | — | 0.07 |
| 20 | 0.02 | 0.04 | 0.02 | — |
| 21 | 0.06 | 0.07 | 0.01 | — |
| 22 | 0.18 | 0.09 | — | 0.09 |
| 24 | 0.12 | 0.11 | — | 0.01 |
| 26 | None | 0.10 | 0.10 | — |
| 30 | 0.08 | 0.07 | — | 0.01 |
| 36 | 0.16 | 0.14 | — | 0.02 |
| Above 36 | None | 0.05 | 0.05 | — |
| Average..... | 0.552 | 0.534 | — | 0.018 |
| If 3- and 4-inch pipes are eliminated from consideration the average figures become: | | | | |
| | 0.334 | 0.325 | — | 0.009 |

* This period excludes the abnormal frost at the beginning of 1929.

The returns from these authorities were carefully analysed and a large number of graphs were prepared, from which the figures in table 2 have been compiled.

The total length of water mains of various sizes owned by these authorities in 1928 was 4,838 miles, and the average number of bursts per mile for all sizes of pipes during each of the two periods is given in table 2.

In preparing table 2, 13-, 16-, 17-, 19-, 25-, 28- and 33-inch mains have been excluded from consideration as they are seldom used.

Comparing each of the two periods as a whole, it will be seen that there is an actual decrease in the frequency of failures in mains of all sizes in the latter five years as compared with the earlier similar period, also that the same remark applies when the smallest sized mains (3- and 4-inch) are excluded.

It should be pointed out that, of the length of 4,838 miles of mains to which the foregoing figures apply, 75.25 percent are of sizes up to and including 6-inch diameter.[2]

From a general examination of the figures in table 2, it is apparent that the liability of water mains to fail decreases as their size increases, and that 3-inch mains are by far the most vulnerable. Comparing the relative frequency of failures in the two periods under review it will be seen broadly that up to and including 9-inch mains there are fewer failures in the latter than in the earlier period. In medium sizes, i.e., from 10- to 21-inch, the reverse is the case, while in the larger sizes above 21-inch, the frequency of failures is less at the present time than it was 15 years ago.

The figures relating to 9-inch mains are rather curious, as the number of failures in mains of this size in both periods is higher than would be expected from comparison with figures for mains of approximately the same size. Table 2 clearly shows that larger diameter mains are much less liable to fail than the smaller sizes and it should be pointed out that from the point of view of the resulting damage, failures of the larger mains are the more serious.

The graph which represents failures over the continuous period from 1910 to 1928 indicates a curious feature, viz., a pronounced rise from about the year 1914 to about 1921. Between these dates the results from a larger number of authorities have been taken into consideration, and to some extent this may explain the increase, if the additional authorities have higher figures. Another explanation may be that during the war period mains were not either laid or managed as carefully as usual. It may also be due to the heavy traffic during the war period, when there was a reduction in the usual work of highway maintenance.

[2] Figures apply to information from 15 authorities—*Editor*.

## 2. EFFECT OF FROST

*Number of failures during the year ended March 31, 1929, as compared with the average per year of the immediately preceding four years.*

The returns received from 32 authorities furnish evidence on which it is possible to base an accurate opinion as to the relative number of failures on water mains during a winter of exceptional severity as compared with other years in which the climatic conditions have been more moderate.

The frost which occurred in February and the early part of March, 1929, lasting for about a month, was one of the most severe and prolonged experienced in this country in recent years, while the four immediately preceding years had more normal winter conditions. When the returns were tabulated, it was found that there were very marked differences in the number of failures per mile on mains belonging to the different authorities.

An analysis of the figures showed that the authorities naturally fell into two classes, and, on further examination, it was found that the class to which any individual authority belonged depended upon the source from which its water supply was derived. Those authorities drawing their supplies from wells, or other underground sources, are found to have a much smaller number of failures than authorities dependent upon surface supplies, and, in the following tables 3, 4 and 5, the two classes are dealt with separately.

Table 3 has been prepared from the information furnished by the 32 authorities whose returns as to the lengths of mains of various sizes have been utilised in the last two columns of table 1.

They have been divided into two classes according to the source from which their supplies are derived. The authorities numbered 1 to 11 obtain their water either wholly or in the main from underground sources, viz., wells, boreholes, springs or collieries, while the authorities numbered 12 to 32 depend chiefly or altogether upon surface supplies from catchment areas, rivers or streams.

It appears clear from table 3 that, whatever be the nature of the source from which the supply is drawn, failures are more frequent during a year which embraces an exceptionally severe and prolonged frost period as compared with a period of years in which more average conditions prevail.

Table 3 shows the average number of bursts per mile for the total length of mains for each authority during the period of four years April, 1924, to March, 1928, compared with the year April, 1928, to

TABLE 3

*Number of failures during the year ended March 31, 1929, as compared with the average per year of the immediately preceding four years*

| NUMBER | AUTHORITY | MILEAGES (PER CENT OF TOTAL) | AVERAGE NUMBER OF BURSTS PER MILE PER ANNUM (ALL SIZES) | | | |
|---|---|---|---|---|---|---|
| | | | Period | | Increase | Decrease |
| | | | Years ended March 31, 1925–28 | Year ended March 31, 1929 | | |
| **Underground supplies (11 authorities)** | | | | | | |
| 1 | Bath | 1.03 | 0.109 | 0.092 | — | 0.017 |
| 2 | Bexhill | 1.81 | 0.078 | 0.105 | 0.027 | — |
| 3 | Coventry | 6.37 | 0.238 | 0.119 | — | 0.119 |
| 4 | Hull | 10.55 | 0.058 | 0.096 | 0.038 | — |
| 5 | Lincoln | 3.38 | 0.083 | 0.075 | — | 0.008 |
| 6 | Nottingham | 15.51 | 0.064 | 0.088 | 0.024 | — |
| 7 | Portsmouth | 9.34 | 0.210 | 0.260 | 0.050 | — |
| 8 | Scarborough | 1.85 | 0.089 | 0.103 | 0.014 | — |
| 9 | Southampton | 15.00 | 0.053 | 0.042 | — | 0.011 |
| 10 | South Staffs | 30.16 | 0.229 | 0.273 | 0.044 | — |
| 11 | West Cheshire | 5.00 | 0.102 | 0.196 | 0.094 | — |
| 11 | Authorities | 100.00 | 0.125 | 0.161 | 0.036 (28.8%) | — |
| **Surface supplies (21 authorities)** | | | | | | |
| 12 | Arbroath | 0.16 | 0.356 | 0.346 | — | 0.010 |
| 13 | Barnsley | 0.69 | 0.473 | 0.548 | 0.075 | — |
| 14 | Birkenhead | 1.18 | 0.182 | 0.277 | 0.095 | — |
| 15 | Birmingham | 6.61 | 0.220 | 0.446 | 0.226 | — |
| 16 | Blackburn | 0.48 | 1.298 | 0.955 | — | 0.343 |
| 17 | Bristol | 3.11 | 0.225 | 0.297 | 0.072 | — |
| 18 | Clydebank | 0.11 | 0.889 | 1.056 | 0.167 | — |
| 19 | Durham | 4.18 | 0.471 | 0.478 | 0.007 | — |
| 20 | Glasgow | 5.32 | 0.649 | 0.730 | 0.081 | — |
| 21 | Greenock | 0.43 | 0.416 | 0.446 | 0.030 | — |
| 22 | Halifax | 1.41 | 0.218 | 0.327 | 0.109 | — |
| 23 | Leeds | 4.02 | 0.954 | 0.342 | — | 0.612 |
| 24 | Leicester | 2.78 | 0.664 | 0.652 | — | 0.012 |
| 25 | Liverpool | 6.62 | 0.514 | 0.674 | 0.160 | — |
| 26 | London | 42.62 | 0.311 | 0.457 | 0.146 | — |
| 27 | Manchester | 7.92 | 0.599 | 0.777 | 0.178 | — |
| 28 | Newcastle | 6.37 | 0.392 | 0.385 | — | 0.007 |
| 29 | Oldham | 1.49 | 0.570 | 0.618 | 0.048 | — |
| 30 | Stockport | 1.91 | 0.228 | 0.187 | — | 0.041 |
| 31 | Swansea | 1.62 | 0.719 | 0.638 | — | 0.081 |
| 32 | Torquay | 0.97 | 0.118 | 0.101 | — | 0.017 |
| 21 | Authorities | 100.00 | 0.415 | 0.499 | 0.084 (20.4%) | — |

March, 1929, and clearly shows the wide variation in the average figures for the two classes. In both classes the number of failures during the exceptionally severe winter is definitely greater than in

TABLE 4

*Number of failures during the year ended March 31, 1929 as compared with the average per year of the immediately preceding four years*

Underground supplies (11 authorities)

| SIZE OF MAIN | MILEAGES (PER CENT OF TOTAL) | AVERAGE NUMBER OF BURSTS PER MILE PER ANNUM | | | |
|---|---|---|---|---|---|
| | | Period | | Increase | Decrease |
| | | Years ended March 31, 1925-28 | Year ended March 31, 1929 | | |
| *inches* | | | | | |
| 2 | 1.72 | 0.297 | 0.290 | — | 0.007 |
| 2½ | 3.70 | 0.416 | 0.444 | 0.028 | — |
| 3 | 25.83 | 0.248 | 0.302 | 0.054 | — |
| 4 | 31.55 | 0.090 | 0.135 | 0.045 | — |
| 5 | 2.64 | 0.125 | 0.120 | — | 0.005 |
| 6 | 13.35 | 0.048 | 0.080 | 0.032 | — |
| 7 | 1.09 | 0.115 | 0.087 | — | 0.028 |
| 8 | 2.19 | 0.026 | 0.014 | — | 0.012 |
| 9 | 2.60 | 0.042 | 0.024 | — | 0.018 |
| 10 | 2.34 | 0.036 | 0.135 | 0.099 | — |
| 12 | 4.14 | 0.065 | 0.038 | — | 0.027 |
| 13 | 0.22 | 0.092 | 0.142 | 0.050 | — |
| 14 | 1.07 | 0.009 | 0.059 | 0.050 | — |
| 15 | 0.77 | 0.023 | 0.124 | 0.101 | — |
| 16 | 0.64 | 0.099 | 0.197 | 0.098 | — |
| 18 | 1.11 | 0.014 | 0.028 | 0.014 | — |
| 20 | 0.99 | 0.055 | Nil | — | 0.055 |
| 21 | 1.03 | 0.018 | 0.024 | 0.006 | — |
| 24 | 2.24 | 0.099 | 0.014 | — | 0.085 |
| 30 | 0.48 | Nil | Nil | — | — |
| 33 | 0.18 | 0.129 | Nil | — | 0.129 |
| 36 | 0.12 | Nil | Nil | — | — |
| All sizes | 100.00 | 0.125 | 0.161 | 0.036 | — |

the preceding years, but while the average number of failures in the case of underground supplies is only one-third of those for surface supplies, the percentage increase during a year of exceptional frost is greater in the former than in the latter class—being nearly 30 percent as compared with 20 percent.

In tables 4 and 5 the figures used in table 3 are arranged so as to show the average number of failures on mains belonging to the authorities in each of the two classes for each size of pipe. These tables show more clearly the influence of the size of main upon the number of failures, and they also show that the increase in the number of failures is mainly confined to the smaller sizes of pipes.

The investigation as to the effect of frost on water mains may be carried further by examining the incidence of bursts during any particular year, but it is difficult to give exact figures as to the number of bursts occurring during frost periods as compared with non-frost periods, as the various authorities from whom returns have been received give widely varying periods during which they consider that frost conditions prevailed.

Taking an average of the records returned, it appears that 58.7 percent of the failures occurred in non-frost periods as compared with 41.3 percent in frost periods. When it is remembered that three out of the twelve months may be considered as a liberal allowance to cover that portion of the year during which frost may be expected, it is clear that frost is an important cause of failure.

In considering the records of the instances of mains fracturing during a period of frost, it is difficult to determine whether the damage would have taken place if the pipes had not been detrimentally affected from some other cause, e.g., electrolysis, graphitic deterioration, etc., but it is quite clear that in the case of the smaller mains, particularly those having rigid joints, such as turned and bored or rust joints (made with sal ammoniac and iron borings) where expansion and contraction are not possible, the fractures are due in great measure to the difference in temperature between the water admitted to the mains and the temperature of the soil.

To illustrate this point, table 6 sets forth all the bursts on the mains of the Metropolitan Water Board during the period from October 1, 1928, to March 31, 1929. The relatively small number in the Kent District as compared with the other districts will be observed. In the Kent District the water is chiefly derived from wells—the temperature of which is about 52°F.

In the Northern District it will be observed how large a number of bursts occurred in mains of 3- and 4-inch diameter. These for the most part have rust joints and are situated in the area of the late East London Waterworks Company, which Company employed the rust joint very extensively; also the water is taken from the open filter beds.

TABLE 5

*Number of failures during the year ended March 31, 1929, as compared with the average per year of the immediately preceding four years*

Surface supplies (21 authorities)

| SIZE OF MAIN | MILEAGES (PER CENT OF TOTAL) | AVERAGE NUMBER OF BURSTS PER MILE PER ANNUM | | | |
|---|---|---|---|---|---|
| | | Period | | Increase | Decrease |
| | | Years ended March 31, 1925–28 | Year ended March 31, 1929 | | |
| *inches* | | | | | |
| 2 | 0.04 | 1.306 | 2.224 | 0.918 | — |
| 3 | 23.65 | 0.758 | 0.959 | 0.201 | — |
| 4 | 38.15 | 0.400 | 0.473 | 0.073 | — |
| 5 | 5.53 | 0.449 | 0.551 | 0.102 | — |
| 6 | 8.35 | 0.270 | 0.283 | 0.013 | — |
| 7 | 3.53 | 0.140 | 0.169 | 0.029 | — |
| 8 | 1.51 | 0.241 | 0.240 | — | 0.001 |
| 9 | 3.22 | 0.295 | 0.336 | 0.041 | — |
| 10 | 1.93 | 0.156 | 0.128 | — | 0.028 |
| 12 | 4.38 | 0.145 | 0.129 | — | 0.016 |
| 13 | 0.04 | 0.485 | 0.448 | — | 0.037 |
| 14 | 0.22 | 0.274 | 0.200 | — | 0.074 |
| 15 | 1.16 | 0.172 | 0.128 | — | 0.044 |
| 16 | 0.02 | 0.400 | 0.400 | — | — |
| 17 | 0.15 | 0.169 | 0.295 | 0.126 | — |
| 18 | 1.36 | 0.074 | 0.095 | 0.021 | — |
| 19 | 0.01 | 0.206 | Nil | — | 0.206 |
| 20 | 0.16 | 0.055 | 0.039 | — | 0.016 |
| 21 | 0.43 | 0.044 | 0.058 | 0.014 | — |
| 22 | 0.04 | 0.077 | 0.154 | 0.077 | — |
| 24 | 1.67 | 0.076 | 0.078 | 0.002 | — |
| 25 | 0.09 | 0.017 | Nil | — | 0.017 |
| 26 | 0.01 | 0.125 | Nil | — | 0.125 |
| 27 | 0.04 | Nil | Nil | — | — |
| 28 | 0.12 | Nil | 0.052 | 0.052 | — |
| 30 | 1.75 | 0.023 | 0.032 | 0.009 | — |
| 33 | 0.02 | 0.099 | Nil | — | 0.099 |
| 36 | 1.37 | 0.056 | 0.059 | 0.003 | — |
| Above 36 | 1.05 | 0.265 | 0.035 | — | 0.230 |
| All sizes | 100.00 | 0.415 | 0.499 | 0.084 | — |

It will be borne in mind that in more recent years it has been the custom to pay greater attention to the depth at which water mains are laid. The subject of depth was particularly considered after the great frost of 1895, when the water was frozen in many miles of mains.

TABLE 6

*Number of failures during the period October 1, 1928, to March 31, 1929, on the mains of the Metropolitan Water Board*

| SIZE OF MAIN | KENT DISTRICT "A" | NORTHERN DISTRICT "B" | SOUTHERN DISTRICT "C" | WESTERN DISTRICT "D" | TOTAL |
|---|---|---|---|---|---|
| *inches* | | | | | |
| 2 | — | 2 | — | 8 | 10 |
| 3 | 99 | 449 | 189 | 172 | 909 |
| 4 | 38 | 422 | 224 | 462 | 1,146 |
| 5 | — | 46 | 41 | 128 | 215 |
| 6 | 5 | 70 | 10 | 15 | 100 |
| 7 | — | 18 | 3 | 9 | 30 |
| 8 | 2 | 3 | 1 | 1 | 7 |
| 9 | — | 5 | 2 | 6 | 13 |
| 10 | 1 | — | 4 | — | 5 |
| 12 | 1 | 1 | 3 | 2 | 7 |
| 15 | — | 1 | 1 | — | 2 |
| 16 | 1 | — | — | — | 1 |
| 18 | 1 | 2 | — | 1 | 4 |
| 22 | — | 1 | — | — | 1 |
| 24 | — | — | 1 | 1 | 2 |
| 30 | — | — | 1 | 1 | 2 |
| 36 | — | — | 2 | 2 | 4 |
| 48 | — | — | — | 1 | 1 |
| Totals..... | 148 | 1,020 | 482 | 809 | 2,459 |
| Bursts per mile of mains of all sizes... | 0.02 | 0.14 | 0.07 | 0.11 | |

*Note.* "A" supplied from wells.

"B" supplied from Rivers Lee, Thames and New River, supplemented occasionally by water from wells.

"C" and "D" supplied from River Thames.

In that year it was recorded in London that the temperature was below 32°F. on 25 days, viz., from January 25, to February 18, which was a record since 1814, in which year the temperature was below 32°F. during 26 days. The temperature of the earth at a depth of 1

TABLE 7

*Temperature*

| WEEK ENDED | MEAN TEMPERATURE AT KEW BRIDGE | TEMPERATURES AT HAMMERSMITH (RIVER SUPPLY) MEAN TEMPERATURE IN DEGREES FAHRENHEIT OF SEVEN DAILY READINGS TAKEN AT 9 A.M. | | |
|---|---|---|---|---|
| | Air (see note) | Temperature of ground 4 feet below surface | Water in mains | Temperature of water compared with that of ground |
| *1929* | | | | |
| January 4 | 32.00 | 47.61 | 41.86 | −5.75 |
| January 11 | 31.71 | 45.36 | 38.22 | −7.14 |
| January 18 | 31.00 | 44.18 | Min. 36.37 | −7.81 max. minus |
| January 25 | 33.43 | 42.86 | 37.93 | −4.93 |
| February 1 | 38.71 | 42.54 | 38.82 | −3.72 |
| February 8 | 36.00 | 43.85 | 42.75 | −1.10 |
| February 15 | Min. 23.85 | 42.93 | 40.29 | −2.64 |
| February 22 | 29.43 | 40.68 | 36.39 | −4.29 |
| March 1 | 31.28 | 39.55 | 37.65 | −1.90 |
| March 8 | 33.71 | Min. 39.00 | 38.29 | −0.71 |
| March 15 | 42.14 | 39.64 | 41.96 | +2.32 |
| March 22 | 40.85 | 40.75 | 43.25 | +2.50 |
| March 29 | 50.00 | 43.00 | 50.07 | +7.07 |
| April 5 | 48.00 | 45.00 | 52.57 | +7.57 |
| April 12 | 43.71 | 45.85 | 50.85 | +5.00 |
| April 19 | 44.71 | 45.67 | 48.57 | +2.90 |
| April 26 | 46.14 | 47.11 | 50.55 | +3.44 |
| May 3 | 45.14 | 47.57 | 50.85 | +3.28 |
| May 10 | 50.85 | 47.57 | 53.00 | +5.43 |
| May 17 | 53.57 | 49.43 | 56.71 | +7.28 |
| May 24 | 57.57 | 51.43 | 60.36 | +8.93 |
| May 31 | 61.85 | 55.64 | 65.50 | +9.86 Max. plus |
| June 7 | 57.42 | 56.71 | 64.64 | +7.93 |
| June 14 | 60.28 | 56.89 | 64.68 | +7.79 |
| June 21 | 62.28 | 58.03 | 66.53 | +8.50 |
| June 28 | 58.57 | 59.64 | 65.93 | +6.29 |
| July 5 | 59.00 | 59.96 | 63.82 | +3.86 |
| July 12 | 61.85 | 60.04 | 64.82 | +4.78 |
| July 19 | Max. 68.85 | 62.45 | 63.04 | +0.59 |
| July 26 | 68.57 | 65.30 | Max. 72.93 | +7.63 |
| August 2 | 62.71 | 65.96 | 69.61 | +3.65 |
| August 9 | 60.21 | 64.07 | 66.93 | +2.86 |
| August 16 | 64.07 | 63.68 | 67.86 | +4.18 |
| August 23 | 61.71 | 64.25 | 68.00 | +3.75 |
| August 30 | 65.07 | 65.14 | 70.00 | +4.86 |
| September 6 | 66.43 | 66.58 | 69.04 | +2.46 |

TABLE 7—*Concluded*

*Temperature*—Concluded

| WEEK ENDED | MEAN TEMPERATURE AT KEW BRIDGE | TEMPERATURES AT HAMMERSMITH (RIVER SUPPLY) MEAN TEMPERATURE IN DEGREES FAHRENHEIT OF SEVEN DAILY READINGS TAKEN AT 9 A.M. | | |
|---|---|---|---|---|
| | Air (see note) | Temperature of ground 4 feet below surface | Water in mains | Temperature of water compared with that of ground |
| *1929* | | | | |
| September 13.... | 68.50 | 67.97 | 71.79 | +3.82 |
| September 20.... | 63.21 | Max. 68.29 | 69.47 | +1.18 |
| September 27.... | 57.79 | 67.21 | 64.79 | −2.42 |
| October 4..... | 58.43 | 66.14 | 64.74 | −1.40 |
| October 11..... | 51.93 | 63.57 | 59.79 | −3.78 |
| October 18..... | 55.00 | 62.00 | 59.53 | −2.47 |
| October 25..... | 47.14 | 60.50 | 56.36 | −4.14 |
| November 1..... | 43.29 | 58.21 | 53.53 | −4.68 |
| November 8..... | 43.00 | 57.29 | 50.21 | −7.08 |
| November 15.... | 42.14 | 54.29 | 49.07 | −5.22 |
| November 22.... | 41.00 | 52.75 | 47.50 | −5.25 |
| November 29.... | 47.50 | 52.18 | 50.65 | −1.53 |
| December 6..... | 46.07 | 52.90 | 50.75 | −2.15 |
| December 13.... | 44.07 | 51.89 | 48.68 | −3.21 |
| December 20.... | 39.21 | 51.43 | 46.57 | −4.86 |
| December 27.... | 43.14 | 48.79 | 43.71 | −5.08 |
| Average..... | 60.16 | 53.68 | 54.57 | +0.89 |

*Note.* The records of the temperature of the air at Hammersmith are not available, and those given are from observations at Kew Bridge. The mean temperature for the week is arrived at by adding together the maximum and minimum readings for each day and dividing by 14.

Figures in last column:

Average of 28 plus figures = +4.99.
Average of 24 minus figures = −3.88.

foot below the surface as recorded in Camden Square fell on 12 consecutive days below 32°F.

In the year 1929 there were 29 days in which the temperature on the ground in London fell below 32°F.

Another cause contributing to the fracture of water mains in cold weather is the number of supply pipes, hydrants and their connection, and other apparatus in which the water lies nearer to the surface of the ground, and these will in a measure account for the fractures of small mains occurring more frequently.

An interesting comparison of the temperatures of the water in the trunk mains and the earth at a depth of 4 feet below the surface at Hammersmith during 1929 is set forth in table 7.

The temperature of the water in the mains varies at different points in relation to the proximity of the exposed source of supply, e.g., the open filter beds.

### 3. EFFECT OF TRAFFIC AND VIBRATION

The returns received from the various authorities show that those mains in which failures occurred were chiefly laid under the carriageway, the percentage of failures in this position being 85 as against 15 on mains laid under the footway, having due regard to the respective length of pipes laid in these respective positions. These figures are deduced from information given by 32 authorities and they would appear to indicate that vibration due to heavy traffic is an important cause of failure of mains.

The proximity of tramways is in many cases a contributory cause.

Whereas in the early part of the year 1929 the attention of the public was particularly directed to the subject of "burst" water-mains, caused by the severity of the frost period and its long duration, water engineers have for some time been considering the causes attending the fracture as distinct from the bursting of water mains.

The effect of damage to a main of large diameter as compared with a small service main, naturally claims the attention of the public, as also does it affect in greater degree the supply of water and the convenience of the public using the highways.

It will be remembered that some of our trunk mains were laid eighty or more years ago, when the heavily-trafficked streets of to-day were little more than country roads. The depth of the mains was shallow, perhaps not more than 2 feet, and the traffic was light, slow and infrequent. To-day the traffic is heavy, fast and of great volume. Then we have the tramways, the concrete foundations of which are in some cases supported upon the water mains, the effect being to transmit to the mains the blow and vibration not only of the tram cars but of the ordinary vehicular traffic.

Another source of vibration affecting the lead joints of the pipes, if not the pipes themselves is the use of pneumatic tools in breaking up the foundations of the roads; this has been found to cause leakage from lead joints.

These matters cause the water engineer to direct his attention to the

depth at which the mains shall be laid, and to their isolation from hard, rigid occasional bearings, or directly superimposed foundations of highway and tramway structures likely to transmit weight and vibration.

### 4. FAILURES DUE TO OTHER CAUSES

Some of the returns make reference to the deteriorating action of the soil and the effect of electrolysis upon the metal of which the pipes are composed.

In the report on burst mains, recently issued by the Metropolitan Water Board, there is an appendix dealing with the subject of electrolysis and the action of the soil upon the pipes. It is abundantly clear that in the case of certain soils, it is necessary to afford special protection to cast iron, and in a still greater degree to steel pipes.

The operations of other undertakers in the vicinity of water mains is frequently a cause of failure of those mains and accounts for a high proportion of major bursts.

This aspect deserves particular attention in our large towns, where many of the operations attendant upon the laying of cables and the inspection for leakage from other mains, and the laying of drains, are carried out by means of tunnelling. Operations of this kind should be inspected by water authorities, otherwise it may be found that the support of their mains has been interfered with and the protective coating damaged. Damage to the coating of a steel main may lead to rapid corrosion.

### 5. HIGHWAY CONSTRUCTION AND ALTERATION

In concluding this report we wish to make clear our full appreciation of the necessity of maintaining the water mains and apparatus in connection therewith in the highest state of efficiency in order to ensure the protection of private property and the convenience of water consumers and persons using the highways.

We would at the same time draw attention to the fact that in recent years it has become the practice of highway authorities to amend the lines of streets and to alter their levels, in many cases to the detriment of water undertakers whose mains have been rendered insecure by such alterations, thereby necessitating the expenditure by them of considerable sums of money in laying their mains in new positions or lowering or raising them to suit the new conditions of the road surface.

We would further draw attention to the fact that some road author-

ities, when reconstructing roads, allow heavy motor lorries to pass over the subsurface of the road after the metalling has been removed, and in some cases also allow the use of a steam or petrol roller on such surface. This entails considerable stress on the joints of water and gas mains.

These facts indicate the desirability of amendments to the Highway Acts for the protection of water undertakers and other statutory authorities using the subsoil of the streets. To this end we counsel increased coöperation with such authorities.

## GENERAL CONCLUSIONS

1. There has been an actual decrease in the frequency of fractures in mains of all sizes during the last five years as compared with a similar period fifteen years ago, but this decrease is slight.

2. Failures are more frequent in a year which embraces an exceptionally severe and prolonged frost period than during one of more average conditions. The increased frequency is principally in the smaller service mains carrying water of low temperature and where the joints are of a kind not permitting of expansion and contraction.

3. The number of fractures per unit length of main is greater in mains conveying water from surface sources of supply than in the case of mains carrying water from underground sources.

4. The greater liability to failure in the case of small size mains in cold weather may be attributed to some extent to the number of supply pipes, hydrants with their connections, and other apparatus in which the water lies nearer to the surface of the ground.

5. Vibration due to heavy traffic is an important cause of the failure of mains, particularly in cases where the cover over the main is shallow. The proportion of fractures in mains under the carriageway is nearly six times as great as that under the footway having regard to the respective lengths of mains laid in the two positions.

6. The proximity of tramways is in many cases a contributory cause of failures.

7. The operations of other undertakers, interfering with or removing the support, account for a high proportion of the "major bursts" or fractures of water mains of large diameter.

8. Excavations for deep basements in building sites abutting upon the streets are a distinct menace to the stability of water mains.

9. Although the returns did not reveal instances of bursts from

air locks, we consider it desirable to refer to the necessity of providing and maintaining efficient air reliefs in the system of water mains.

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# PROBLEMS OF GROUND-WATER SUPPLY IN FLORIDA[1]

By David G. Thompson[2]

Florida has an abundant water supply, from both surface and underground sources, as is shown by the presence of hundreds of lakes, large springs, wells of large yield, and, in some localities, wells that flow naturally.

Several well-known springs, including Silver Spring and Blue Spring, are among the largest in the United States. The measured flow of Silver Spring has been as much as 822 second-feet.[3] The measured flow of the Silver River, which is formed by the discharge of this and other springs, has ranged from 674 to 1,170 second-feet. The larger quantity is more than half a million gallons a minute, or more than 720 million gallons a day, a quantity sufficient to supply a city the size of New York. The largest springs in Florida are rivaled in size and spectacular features probably only by springs flowing from volcanic rocks in certain states of the Northwest. One of the most striking series of such springs are the Thousand Springs,

[1] Presented before the Florida Section meeting, April 2, 1931. Published by permission of the Director, United States Geological Survey, and of the State Geologist of Florida. A somewhat similar paper has been released by Florida State Geologist for limited distribution within State.

[2] Geologist, Division of Ground Water, U. S. Geological Survey, Washington, D. C.

[3] U. S. Geol. Survey Water-Supply Paper 27, p. 45, 1899; Water Supply Paper 102, pp. 274–275, 1904. From the description given in Water-Supply Paper 102 it appears that the measurements of 822 second-feet included not merely the main boil but also the flow of several springs near the head of the Silver River. A flow of 620 second-feet for the main boil of Silver Spring was measured by D. S. Wallace, district engineer, U. S. Geological Survey, on August 4, 1930. From records of gage heights it is known that at times the flow of the spring has been much larger than this. On February 27, 1917, the flow was only 342 second-feet. The minimum observed flow of the Silver River was 674 second-feet on February 27, 1917, and the maximum 1,170 second-feet on August 4, 1930. The second largest group of springs in Florida which has been measured is Blue Spring Creek, with a maximum recorded flow of 847 second-feet on February 8, 1907, and the third largest group are on Ichatucknee Creek near Hildreth, Suwannee County, with a maximum recorded flow of 467 second-feet on January 30, 1929.

which issue from the walls of lava rock along the Snake River in Idaho.

A well at a phosphate plant in Polk County yields one of the largest well supplies in the world, being pumped at the rate of 8,000 gallons a minute. Many municipalities and industries use ground water, and almost the entire rural population of the State is wholly dependent upon such supplies.

To give some idea as to the importance of its ground-water resources to the State the following figures in regard to the public supplies may be cited. According to statistics furnished by the State Board of Health there are in Florida 175 public water supply systems, all but 12 of which obtain their water from underground sources. It is significant that by far the greater number of communities are supplied with ground water. From incomplete data it is estimated that the total consumption of ground water in the State is about 65,000,000 gallons a day, and of surface water about 20,000,000 gallons, of which a single supply—that of Tampa—accounts for about 12,000,000 gallons.[4] It is estimated that the total cost to the consumers of ground water supplied by waterworks is more than $3,500,000 a year. When the large consumption of water from privately owned wells for domestic, industrial, and irrigation uses is considered it is obvious that the value of ground water to the State is very great.

In spite of the general abundance of ground water in Florida, the development of satisfactory supplies for different purposes has involved many difficult problems, and as the use of water increases with the future development of the State and problems of obtaining water supplies of satisfactory quality will become still more complex and difficult. These problems relate to the quantity of water available, its quality with respect to a variety of uses, and the best means of recovering it and protecting it from pollution. All these problems involve a study of the geologic conditions that govern the occurrence of the ground water.

[4] This and the following statements are based on a compilation of data in "Water supply statistics of American municipalities," second edition, published by The American City Magazine Corporation, 1929. The statistics in that publication relate to only 69 supplies in Florida, but the consumption for the remaining supplies, which are mostly small, was estimated according to population on a conservative basis.

COÖPERATIVE STUDIES BY FEDERAL AND STATE GOVERNMENTS

Studies by the United States Geological Survey in many parts of the country show that where there have been large developments of ground water there has generally been a depletion of the natural supply, as indicated by a lowering in the water level in wells, usually with a decline in the yield of the wells. This has happened even where wells originally flowed under high pressure or yielded so much water by pumping that the supply seemed inexhaustible. In North and South Dakota, for example, the artesian pressure in many wells drawing from the Dakota sandstone originally ranged from 100 to more than 200 pounds to the square inch,[5] the latter pressure sufficient to put water by natural flow to the top of a 40-story building; but the pressure of many of these wells has decreased to almost nothing, and some have ceased to flow. In the vicinity of Chicago the hydrostatic head on the ground water in certain formations dropped from a point 80 feet above the surface in 1864 to more than 250 feet below the surface in 1924.[6] Many other examples could be cited.

It may be expected that similar depletion will occur in parts of Florida, if excessive quantities of water are withdrawn from the underground reservoirs in localities of large cities or irrigation farming. Furthermore, heavy draft may cause water of poor quality to be drawn in. In Florida, more than in most other parts of the country, the problems relating to the quantity of water available are closely connected with those relating to the quality of the water. Much of the water comes from limestone that contains large cavities through which sewage or other contaminating substances can easily move for considerable distances. A very large part of the State is not far from extensive bodies of salt water, and salt water probably lies beneath the entire peninsula at a greater or less depth. The withdrawal of water from wells upsets a natural balance between the salt and fresh water, with the result that wells originally fresh may become salt.

[5] Meinzer, O. E., and Hard, H. A., The artesian water supply of the Dakota sandstone in North Dakota, with special reference to the Edgeley quadrangle: U. S. Geol. Survey Water-Supply Paper 520, pp. 81–84, 1925; U. S. Geol. Survey Bull. 801, pp. 58–61, 1929. Simpson, H. E., Geology and ground-water resources of North Dakota: U. S. Geol. Survey Water-Supply Paper 598, pp. 115–116, 1929.

[6] Anderson, C. B., The artesian waters of northeastern Illinois: Illinois Geol. Survey Bull. 34, pp. 93–94, 1919; Swift & Co., communication to the writer, August, 1924.

Much valuable information in regard to the ground-water supplies of Florida has been collected in past years and published in reports of the State Geological Survey and the United States Geological Survey.[7] The information in these reports relates largely to the distribution of water-bearing beds, the depth at which the beds were struck, the yield of wells, and the quality of the water. Since the publications of these reports there have been new developments of water supply, and the consumption from certain existing supplies has been greatly increased. In localities where large quantities of water have been withdrawn the hydrostatic head of the water, as indicated by the water level or artesian pressure in the wells, has decreased. In some localities near the coast the decrease in head has permitted salt water to enter the wells and to contaminate the fresh water supply to such an extent that it has been necessary to abandon some wells and seek new supplies in different localities. In other localities where contamination by salt water has not yet occurred there is danger that such contamination may occur in the future.

In 1930 Herman Gunter, the State geologist, enlisted the coöperation of the United States Geological Survey in a project to investigate the ground-water resources and problems of the State, especially with a view to determining the safe yield of the water-bearing beds in localities where such supplies are particularly valuable. Arrangements were made for a coöperative investigation to be financed by equal contributions from the Federal and State organizations.

The present investigation was begun in the summer of 1930. It is in charge of the writer, working in close touch with the State geologist and with O. E. Meinzer, geologist in charge of the division of ground water of the United States Geological Survey. The field and office work have been done principally by V. T. Stringfield, an assistant geologist in the Federal Survey, but interpretations of some of the geologic data have been made by Gerald M. Ponton, assistant State geologist.

[7] Matson, G. C., and Sanford, Samuel, Geology and ground waters of Florida: U. S. Geol. Survey Water-Supply Paper 319, 1913 (out of print). Collins, W. D., and Howard, C. S., Chemical character of waters of Florida: U. S. Geol. Survey Water-Supply Paper 596, pp. 177–233, 1928. Sellards, E. H., A preliminary report on the underground water supply of central Florida: Florida Geol. Survey Bull. 1, 1908. Sellards, E. H., and Gunter, Herman, Artesian water supply of eastern and southern Florida: Florida Geol. Survey Fifth Ann. Rept., pp. 103–290, 1913; also other data in reports of the State Geological Survey.

The problem of determining the safe yield of the water-bearing beds in different parts of the State is of so great magnitude that it can not be completed in a short time. Observations in regard to such factors as the effect of seasonal changes in precipitation and consumption must extend over a period of years. The plan adopted has been to begin observations in a few areas where information is most needed. As the survey in these areas is advanced the work can be extended to other areas, while observations of artesian pressure and discharge in flowing wells and of water levels and pumpage in nonflowing wells can be continued in all the areas.

In the summer of 1930 observations were begun in the vicinity of Jacksonville; Orlando, and St. Petersburg, and in an extensive area that includes parts of Manatee, Sarasota, and Charlotte Counties. The work was of necessity largely of a preliminary nature. Information has been obtained in regard to the location of wells, their depth and appropriate yield, the depth and character of the water-bearing beds, the quality of water, and the general geologic conditions that govern the occurrence of the ground-water supply. Periodic measurements were made in numerous observation wells, and automatic water-level recorders or pressure gages, which give a continuous record of changes in water level or pressure, were installed on five wells.

In order to obtain an adequate basis for estimating the quantity of water that can safely be drawn from the water-bearing beds it will be necessary to obtain information on the relative amounts of rainfall that are disposed of by direct run-off through the streams, by evaporation from the soil and transpiration by plants, and by percolation into the ground. It will also be necessary to run lines of levels to many observation wells so that the height of the water in them with reference to sea level may be accurately determined.

## NATURE OF GROUND-WATER PROBLEMS

It is too early to draw any conclusions from the data collected thus far. It is the purpose of the present paper merely to indicate the character of some of the problems and methods of attack, largely by reference to the investigation of similar problems in other parts of the United States.

At the outset it is desirable to understand certain general factors that govern the occurrence of ground water in Florida. The safe yield of a water-bearing formation may be defined as the rate at which water may be drawn from the formation for an indefinitely long period

without depletion of the supply to such an extent that withdrawal at this rate is no longer feasible. The definition involves, besides the element of quantity of water, the elements of time and cost of pumping. In some regions it also involves the quality of water, especially in parts of Florida, for, if the water is withdrawn at too high a rate salt water or water polluted with bacteria may be drawn in.

The principal factors that determine the quantity of water that may be drawn from a given formation are the quantity of surface water available to recharge the formation—that is, to replenish the supply as water is pumped from it; the area of the intake zone in which surface water gains access to the formation; the permeability of the formation, which is its ability to transmit water; and the volume of pore space or other openings in the rocks in which water can be stored for future use.

In Florida the quantity of surface water available is fairly large. The average annual rainfall ranges from 45 to 60 inches, and the topography and soil conditions are conducive to percolation into the ground rather than to rapid run-off over the surface.

The geologic formations of Florida consist principally of clay, sand, gravel, marl, and limestone. Although water is probably present in practically all the geologic formations penetrated by wells in Florida, certain formations are recognized as being the principal sources of supply.

The principal water-bearing formations in Florida consist of beds of limestone. Some of the limestone contains pores through which the water can pass, but some is hard and dense. In both types, however, there are numerous openings from a few inches to many feet in diameter, which have been formed by solution of the rock. These solution channels carry the water very freely, and much water can be stored in them. They give rise to such large springs as Silver Spring, Blue Spring, and many others in the State. Likewise the yield of certain large wells comes from solution channels penetrated by the wells. As the solution channels from which the large springs flow are submerged it has not been possible to explore them. It is not unreasonable to believe, however, that before its emergence the water of some of them flows as underground rivers in caverns comparable to the Mammoth Cave of Kentucky and well known caves in Virginia, Alabama, Tennessee, and other limestone regions where the geologic history has been favorable to the development of solution channels.

The principal visible evidence of the widespread existence of solution channels of considerable magnitude consists of the numerous sink holes that are found in certain parts of the State. These sink holes are places where the rock roofs over the caverns have caved in. To indicate the effectiveness of the ground water in dissolving the limestone and forming caverns and sink holes it is only necessary to state that from analyses of the water of Silver Spring it is is estimated that about 600 tons a day of dissolved material are carried away when the rate of flow of the spring is 825 second-feet.[8]

The very conditions that make the limestone formations of Florida so valuable as water-bearing formations also create other conditions that are unfavorable for the development of the water for domestic, industrial, and irrigation uses. Because of the channeled character of the rocks, with numerous openings to the surface, disease producing organisms can easily reach the water in the limestone and can move considerable distances within a short time. This is in contrast to the filtering action found in sand and even in gravel. Also, because of the freedom with which the water can move through the solution channels, contamination by salt water from the ocean may result more easily with the withdrawal of water by artificial means than where the water-bearing formations consist of sand or gravel interbedded with clay.

In the peninsula of Florida two principal water-bearing formations have been recognized, the Tampa limestone and the Ocala limestone. Areas in which these formations crop out are shown on the geologic map of the State published by the Florida Geological Survey. Both are known to extend under the large areas where they are covered, to depths ranging from a few feet to several hundred feet. The Ocala limestone underlies the whole of the Florida Peninsula, but in some places it is covered by as much as 500 feet of younger rocks. The Tampa limestone is not so widely distributed, but like the Ocala it is in places covered by other formations. Where the rocks are buried and are encountered only in drilled wells their age can be determined by a study of the fossil remains contained in the drill cuttings, some of which are so small that they must be studied microscopically. The correlation of the several formations in dif-

[8] Sellards, E. H., A preliminary report on the underground water supply of central Florida: Florida Geol. Survey Bull. 1, 1908, p. 47; Some Florida lakes and lake basins: Florida Geol. Survey Third Ann. Rept., 1910; Sixth Ann. Rept., 1914.

ferent parts of the State by means of the identification of these organisms in cuttings from drilled wells has constituted a phase of the work of the State Geological Survey which is of much value in studies of the ground-water supplies.

Much water enters the Tampa and Ocala limestones in the areas where they crop out, but it is less certain whether much water from the surface reaches them in areas where they are covered by other materials. Observations show that in at least two localities downward percolation, if it occurs, is not rapid, for the water level in deep wells tapping the principal water-bearing limestone stands several feet below the water level in near-by shallow wells or lakes.

Where recharge does not occur locally by downward percolation but the water moves in from distant recharge areas, the hydraulic principles involved are different from those where there is local recharge. It therefore will be necessary to make studies to determine the actual conditions not merely in the comparatively small areas where the water is being used but also in the more distant intake areas.

## LOCAL PROBLEMS

A somewhat unusual problem relating to the utilization of the ground water in Florida arises from the use of wells for drainage of surface water. Because of the open character of the rocks, through which water can move easily, large quantities of water can be poured into the wells in localities where the water level in the formation is below the surface of the ground. In certain areas where good surface drainage is lacking wells have been drilled for the specific purpose of draining the surface run-off and even the sewage into the ground. The principal area in which this method of drainage is used is in Orange County. Practically all the run-off from rainfall and all the sewage of the city of Orlando is disposed of through about 90 wells which have been drilled by the city. About 30 wells have been drilled by the Orange County highway department for drainage of country roads, and several privately owned drainage wells are also in use.

The drainage wells penetrate principally the Ocala limestone. They are from 6 to 16 inches in diameter and from 160 to more than 800 feet deep. The capacity of the wells to put water into the ground is as much as several thousand gallons a minute. In wet seasons large quantities of water are poured into the wells.

In recent years prior to 1930 the capacity of the drainage wells

generally was sufficient to dispose of the excess surface waters, and the results were considered to be highly satisfactory. In 1930, during a period of unusually heavy rainfall, the wells in certain parts of the area failed to draw off the water fast enough, and low areas were flooded. An exceptionally serious condition occurred in a part of Orlando, where flood water mixed with sewage covered city streets and partly surrounded residences.

It was at first thought that the trouble was due to lack of sufficient wells to carry off the water and perhaps in part to the clogging of some of the wells. However, certain observations suggest that the flooding was due to the inability of the water-bearing formations to carry off the water as fast as it was supplied. This caused the water level in the formations to rise higher than usual, and in certain low tracts it seems to have risen so high that water flowed out of the wells. A recurrence of flood conditions similar to those in 1930 may be expected whenever there are heavy rains. In fact, it is conceivable that they may become more frequent as increased construction of houses and streets, with increased drainage facilities, brings about a greater concentration of run-off in a short time after the precipitation occurs.

The problems of the Orlando area are of two kinds—one relating to the quantity of water that can be drained into the water-bearing formation and the other to the danger of pollution of water in the formation that will be used for domestic purposes in other localities.

The possibility of pollution deserves serious attention. Polluted water can move rapidly and for considerable distances through the solution channels in the limestone. Tests in an area in Tennessee, where the water-bearing formation contained solution channels as in Florida, showed that water moved about 5 miles in six days. It is not unreasonable to suppose that such a rate might be attained in the Orlando area.

The State Board of Health has taken notice of the danger of polution from drainage wells and has set up regulations prohibiting the construction of such wells with a certain distance of wells used for domestic supplies; and it has kept a careful watch for any signs of pollution. In spite of these precautions polluted water may suddenly appear in unexpected places. Studies are therefore needed to determine the direction in which the drainage water from the Orlando area is moving, the rate of movement, and the possibility of transmission of pollution for long distances.

The problems in the Jacksonville, St. Petersburg, and Manatee-Sarasota areas relate primarily to the danger of contamination of the beds containing fresh water by salt. These problems, however, in turn depend upon the rate of recharge of the formations and the freedom with which water percolates through them.

In the Jacksonville area a considerable quantity of water, estimated at about 25 million gallons a day, is obtained from flowing wells drawing from the Ocala limestone. In the Manatee-Sarasota area some 600 wells, most of which flow more or less, are used for irrigating truck crops and citrus fruits. In both areas, as consumption of water has increased there has been some delcine in the artesian pressure—between 30 and 40 feet in the area of heaviest draft in Jacksonville, and 10 feet or more in the Manatee-Sarasota area. The question arises whether there may be further decreases in the artesian pressure, even to the point where it may be necessary to pump the water from the wells. A question of equal or greater importance is whether there is any danger of salt water being drawn into the formation if the artesian pressure continues to be lowered. The center of draft from the wells in the Manatee-Sarasota area is much nearer the ocean than that in the Jacksonville area, and the danger of contamination by salt water may be greater there. Water containing moderate amounts of sodium chloride is obtained from wells near the ocean in the Manatee-Sarasota area.

Certain general relations between fresh water and salt water along seacoasts are well known.[9] The fundamental facts to be understood in regard to the relation between fresh ground water and salt water near seacoasts are these: (1) Where bodies of fresh and salt water are in contact, the salt water, being heavier than the fresh, will lie below the fresh water—in other words, the fresh water floats, so to speak, on the salt water. (2) The depth to the contact between the fresh and salt water is a function of the height of the water table above sea level and of the density of the salt water. This may be expressed by a simple formula:

$$h = \frac{t}{g - 1}$$

[9] These are summarized by Brown, J. S., A study of costal ground water with special reference to Connecticut: U. S. Geol. Survey Water-Supply Paper 537, 1925; Relation of sea water to ground water along seacoasts: Am. Jour. Sci., 5th ser., vol. 4, pp. 274–294, 1922.

in which $h$ is the depth of the fresh water below sea level; $t$ is the height of the fresh water or hydrostatic head above sea level, and $g$ is the density of the salt water. The density of ocean water is between 1.025 and 1.030. This means that if the top of the fresh water is between 25 and 30 feet above sea level, the fresh water will extend about 1,000 feet below sea level. In other words, for each foot of fresh water above sea level the fresh water extends between 30 and 40 feet below sea level.

From these relations it is found that in an area surrounded by salt water, like the peninsula of Florida, sea water may be present at a greater or less depth beneath the entire area. The maximum head of fresh water above sea level on the peninsula, according to the available information, is estimated to be about 85 feet. Accordingly, the depth to salt water is presumable nowhere more than 2,800 to 3,500 feet, depending upon the density of the salt water. Of course, in many places the depth to the salt water is very much less than these figures.

In this connection three facts may be emphasized: (1) In the developing of ground-water supplies on the peninsula of Florida care must be taken not to drill too deep, lest salt water be drawn in. (2) The artesian head in some places along the coast may be due entirely to back pressure of the salt water in the ocean, consequently, flowing wells are not a sign that contamination by salt water is unlikely. (3) A lowering of the hydrostatic head by pumping or by free flow from wells under artesian pressure may result in contamination not merely in wells near the ocean by lateral movement of salt water, but also at points distant from the ocean by upward movement from the body of salty ground water that underlies the peninsula.

The city of St. Petersburg is among the places where contamination by salt water has occurred as the consumption from its wells has increased. In 1930 the city completed arrangements with a water company whereby a new supply of water has been obtained from a newly developed well field in the so-called Cosme-Odessa region, about 30 miles north of St. Petersburg. The site of the well field was chosen after a study by the consulting engineers of the hydrologic principles involved, in order to obviate so far as possible the danger of contamination by salt water. The capacity of the wells in the field is considerably in excess of the present consumption of St. Petersburg, and it may be assumed that the needs of the city can be met without difficulty for some years to come.

Although there appears to be no pressing problem in the St. Petersburg area, a study of conditions there has been undertaken in the present investigation because it is believed that information in regard to certain fundamental hydrologic principles, which will be valuable in the studies of ground-water conditions in other parts of Florida, can thus be obtained. The opportunity is offered to investigate not only the condition of contamination by salt water of wells in the city, but also to determine whether since the old wells have been shut down, there has been a return to the conditions existing before contamination occurred. There is also opportunity for a study of the effect of pumping in a well field from the very beginning of its development, and furthermore in a field that is not affected by pumping in other areas.

In certain localities in Florida salt water has been encountered in deep wells drilled in search for oil. These wells have been allowed to flow freely, and there is evidence that water from at least one deep well has leaked into overlying fresh water formations and has contamined them for a distance of a few hundred feet from the well. It is highly desirable that such wells be plugged to prevent any further contamination. Legislation has been enacted authorizing the State geologist to take steps to prevent the waste of water from flowing wells in Manatee, Sarasota, and Charlotte Counties, and also to prevent contamination by salt water. This law should be strictly enforced, and should be extended to cover the entire State.

## FLUCTUATIONS OF HYDROSTATIC HEAD

A valuable source of information in each of the current investigations in different parts of Florida is the determination of the relation between fluctuations of the hydrostatic head on the water in the ground, as indicated by the water level in wells, and changes in the rainfall and in the consumption from the wells, or, in the drainage wells, in the quantity of water poured into the ground. The hydrostatic head may be affected by a number of factors, such as rainfall, stream flow, evaporation and transpiration, atmospheric pressure, draft from wells, tides, and even by earthquakes and the passage of railroad trains. Owners of wells frequently state that the water level in their locality never changes. In many areas it is only by means of automatic water level recorders that changes in head may be determined. However, investigations in many regions show that the head everywhere is fluctuating almost continually. In order

that the amounts of fluctuation of head from one season to another due to certain critical factors may be compared, it is necessary that the different types of fluctuation be understood.

One of the most common types of fluctuation of the hydrostatic head is that due to pumping. It should be understood that the same results follow the withdrawal of water from flowing wells as from wells that are pumped. When pumping starts the head drops not only in the well, but for some distance around. The rate of drop is at first rapid and then becomes slower and slower until eventually, if pumping continues long enough, a state of equilibrium is reached. When pumping ceases the water rises at first rapidly and then at a decreasing rate. The amount of fluctuation and the extent of the area in which it occurs differ according to the thickness, areal extent, and permeability of the water-bearing formation, and the quantity of water pumped. The amount of fluctuation in observation wells also differs according to the distance from the pumped well to the observation wells. By a study of these factors it is possible to obtain some idea though perhaps only relative, of the effect of future changes in the rate of withdrawal of water.

If the water-bearing formation is permeable and capable of yielding large quantities of water the observed fluctuation in the observation well may be only a few inches or a few feet, depending on the distance from the observation well to the pumped well or wells. If the formation is less permeable the fluctuation for the same amount of pumping may be many feet.

In localities where a number of wells are being pumped the observed fluctuation of head is the composite result of the effect of all the wells. In such localities the consumption during the day is generally greater than it is at night, so that a daily fluctuation of head occurs. For similar reasons weekly and seasonal fluctuations are found in many regions.

Imposed on the fluctuations of head due to pumping there are in many regions fluctuations due to other causes, and to evaluate properly the fluctuations due to pumping the effect of these extraneous fluctuations must be understood. In some localities there have been observed fluctuations of head due to changes in atmospheric pressure as storms and centers of fair weather passed over the country. In the Grand Prairie region of Arkansas such fluctuations, coinciding closely with the change in barometric records of the Weather Bureau, but with the direction of movement reversed, have been observed

almost simultaneously in wells scattered over a distance of 55 miles.[10] The ratio between the fluctuations of the wells is practically the ratio of the densities of mercury and water—in other words, the wells constitute huge natural barometers. The fluctuations of head due to atmospheric pressure in the Grand Prairie region have amounted to as much as 8 inches in a day or two. As fluctuations of less than this amount during pumping tests were significant it was necessary to correct the observations for the barometric fluctuations.

In certain parts of the same region in Arkansas fluctuations of head of as much as 35 feet coincided rather closely with changes in the level of a near-by river. On the other hand, wells in another part of the same region and just as close to the river showed no such fluctuation. These differences indicated some difference in the geologic conditions in the two parts of the region.

In some localities there is abundant evidence that the hydrostatic head is affected by rainfall, but in others there is no indication of such influence. It is difficult at times, however, to determine whether a given rise in head is due to the rainfall percolating to the water-bearing bed, or whether the same result is attained merely because the rain has caused a decrease in consumption of water for irritation, domestic use, or other uses that are influenced by weather conditions. The effect of rainfall is likely to be not very noticeable in formations under artesian pressure, especially at places some distance from the intake area of the formations. Where artesian conditions do not exist and water can percolate directly to the water table, the head may respond rapidly to rainfall if the water table is near the surface but more slowly if it lies deep below the surface. Where the available open space in the formation is relatively small a given depth of rainfall may produce a much greater rise in head. Thus in a shallow well near Old Bridge, N. J., the water rose about 1 foot for each inch of rain, indicating an effective porosity of less than 10 per cent. The water in this well began to rise almost as soon as the rain began and stopped when the rain stopped.

It is well known that in many localities near seacoasts the hydrostatic head is influenced by the tides, rising and falling with a periodicity similar to that of the tides. The relation between the amount of fluctuation of the tides and the head between times of high and low

[10] Ground-water supplies for rice irrigation in the Grand Prairie region, Arkansas: U. S. Dept. Interior Press Notice 49844, January 26, 1931; also report on same subject by David G. Thompson, in preparation.

tide and of high and low head in the water-bearing formation differ from place to place. In the Atlantic City region, N. J., the fluctuation of the head is about 50 per cent of that of the tides, and the periods of high and low head occur within a short time of the corresponding periods of the tide.[11] Because of certain unusual fluctuations and certain geologic conditions in the region it was concluded that the tidal fluctuations of head in the water-bearing formation are not transmitted laterally from the outcrop of the formation, which presumably is many miles offshore, but that they are due to the alternate compression and expansion of the water-bearing formation as the result of the loading and unloading as the tides rise and fall.

The fluctuation of head due to tides in an observation well in the Atlantic City region was as much as 3 feet in a single day, and much more than that over periods of a few days. In order to obtain the true story for comparing the head on corresponding dates in successive years it is necessary to calculate the average of the two high and two low levels of water in the observation well for each day, as obtained by a water-level recorder. If only occasional measurements were made and these happened to be made in opposite phases of the tide it would be possible to have observed maximum and minimum points during a year several feet higher or lower than the true average levels. This could easily lead to an error of at least 50 per cent in the interpretation of the results.

The fluctuations of head due to a given cause may be partly or wholly masked by fluctuations of a greater magnitude due to other causes. Also a combination of two conditions, either of which would cause a change in head, if they are working in opposite directions and are of equal magnitude, may result in a stationary head. Thus, in a well in Atlantic City at certain times a more or less typical tidal fluctuation is observed; but at other times changes in the rate of pumping from near-by wells during one of the semi-diurnal tide periods partly counteracts the tidal effect, with the result that the curve of fluctuation shows alternate swings or waves of much smaller amplitude than is normal for the true tide curve; and at still other times a different combination of tide and pumping conditions completely obliterates one full swing of the tide curve, and there is no indication of any tidal effect such as is suggested by a lengthy period of no change in head.

[11] Thompson, D. G., Ground-water supplies of the Atlantic City region; New Jersey Dept. Conservation and Development Bull. 30, pp. 55–59, 1928.

Because of the variety of factors that may produce fluctuations in head it is not always easy to determine just what factors are operative or what may be their relative importance. It may be only when some fortuitious combination of conditions masks the effect of certain factors that some other unsuspected factor is revealed. In the Atlantic City region the clue to the explanation of the cause of the tidal fluctuation in deep wells was obtained only at certain times when unusual conditions of wind produced irregular tidal fluctuations in the ocean and wells alike. In the same region it was suspected that changes in rainfall in the intake area of the water-bearing formation produced little if any effect on the hydrostatic head at Atlantic City. However, definite proof of this was not obtained until five years of practically continuous records showed that in 1930 the hydrostatic head rose to higher levels than in the preceding year, in spite of the fact that there was an accumulated deficiency in precipitation greater than at any other time in the period covered by the investigation. The rise in head is believed to have been due to the fact that the financial depression resulted in a considerable falling off in the business of the popular resort, with a consequent decrease in the consumption of water, which is largely obtained from underground sources.

The investigations of recent years in regard to fluctuations of ground-water level and their significance show clearly that if the superintendent of a waterworks using ground water wishes to know the story about the reserve supply of his raw material he must obtain accurate information in regard to fluctuations of the hydrostatic head of the water and determine what causes such fluctuations.

# STAND-BY CHARGES FOR WATER WORKS[1]

By C. E. Thatcher[2]

The subject of this paper will first be defined. The term, "Stand-by Charges for Water Works," shall be understood to mean a charge for "readiness to serve" or "stand-by" service available to municipalities for fire purposes.

Only a few years back little, if any, thought was given to the distribution of costs and rate making in the operation of water works. Generally, those who had to do with the management and operation were interested principally in gross revenue sufficient to pay for the cost of money, operating expense, up-keep, depreciation and, if possible, the establishment of a small reserve fund. To accomplish this the practice was to fix a rate applicable to service rendered to domestic and industrial consumers. Some revenue was derived from a charge for fire hydrants and water used for street sprinkling and sewer flushing.

Water works were developed as towns sprung up. Many towns grew fast in population and later on became real cities. The first installation of water works pipe and equipment was, as a rule, only sufficient to take care of the needs at the time. Rapid growth and modern building construction occurred, fire protection had to be provided and those in charge of water utilities endeavored to keep pace with the growth and development and meet the demands made upon them. The increased costs in operation were met by increasing the rate charged for service rendered to domestic and industrial consumers.

A constant increase in a service or a commodity results in dissatisfaction on the part of the users. Public utility commissions were appealed to by consumers for relief. This occurred not so very long ago. The financial structure of public utilities was looked upon with suspicion. Hearings were held and investigations made. With but few exceptions it was found that water works systems needed all the

[1] Presented before the California Section meeting, October 30, 1930.

[2] Manager of Commercial Department, East Bay Municipal Utility District, Oakland, Calif.

revenue they were receiving. It was found, too, that much of the increase in cost of operating water works was due to a greater amount of interest being paid on money spent for the installation of larger mains and other equipment to provide municipalities with facilities for fire purposes. These enlarged facilities also resulted in considerable increase in plant operation.

Public utility commissions and those having to do with water works management then began to deal with the problem of rate making. Studies were made of the various classes of service rendered by water utilities for the purpose of making an equitable spread of the costs among those receiving the benefit of the services.

If the operation of a water works was confined only to the rendering of service to domestic and industrial consumers, the transmission and distribution mains and other equipment required for such service would be smaller in size than that required to provide additional service to municipalities for fire purposes. It seems reasonable to assume, then, that anyone familiar with water works operation can and should appreciate the fact that, in providing equipment sufficient in size to render additional service to a municipality for fire purposes, is the rendering of a service that should be classed as distinct and separate from other services and the cost and charge for such should not be combined with the cost and charge for any other service. Those who receive the benefit of the service should pay for it. The cost of this special service is, it is thought, a just and proper charge against the municipalities.

A municipality is supported by its tax payers. Therefore, the tax payers should pay for this "readiness to serve" or "stand-by" service. For example: The tenant in a rented house should only pay for the cost of water served to him for domestic use. The owner of the house, the tax payer, should pay the municipality for the protection given to the house against fire. Further, the large area in cities upon which immense buildings and other structures stand must have fire protection. To afford such protection requires the installation of facilities greater in size than would ordinarily be installed for domestic and industrial use only. It follows then that the municipality should pay the water utility for the additional service that it is asked to provide and secure the amount required to pay for such service by a tax levy. The regular consumers of water should be relieved of the burden of paying the cost of this class of service.

It should be appreciated how difficult it is to distribute equitably

the burden of the cost of water served to each class of consumers. There appears to be no recognized authority on distribution of fire service cost. It is an innovation in cost of service distribution and rate making, in water works management, that can be classed as new. Whenever anything new is introduced for use, either a service or a commodity, someone or some agency has to carry on the experimental work and do the pioneering.

A few years ago the Railroad Commission of the State of Wisconsin came to the front and strongly advocated the establishing of a charge against municipalities for fire service facilities and classed such as a "readiness to serve" or "stand-by" service. Naturally, the proposal of such a classification of service and the fixing of a charge covering the cost thereof met with strenuous opposition by the municipalities who were suddenly called upon to shoulder the burden of the cost. The opposition cry was "You are trying to rob Peter to save Paul." It was further contended that the ordinary user of water was, as a rule, the tax payer and should stand the burden. Then, too, the making of a tax levy would upset the tax program and interfere in many localities with the tax rate limit. The Commission assumed a firm stand and the records show that many decisions rendered provide for a charge against the municipalities for fire service facilities. By referring to these decisions it will be found that the amount derived from the charge fixed for fire service averages 33 per cent of the gross revenue of the water utility. This would indicate that the Wisconsin Commission places a high value upon such service and was quick to apportion a cost accordingly against those receiving the benefit of the service. Other public utility commissions have allowed a charge for fire service facilities. The charge though, in nearly every case, yields a lower percentage of gross revenue than does the Wisconsin Commission's charge.

Coming nearer home, California, it is found that the California State Railroad Commission had before it a few years back the problem of fixing a charge against municipalities for fire service. This Commission stated:

"In order to furnish adequate protection against fire it is necessary to install facilities of greater capacity than are required to supply ordinary industrial and domestic demands. Such protection is a direct benefit to the Community and the excess capacity of the Water system which may be deemed justified by the necessity of providing for such emergency demands cannot fairly be charged against regular consumers and paid for under cover of an increase in the unit rates for water consumed for industrial or domestic purposes."

In providing for a charge against municipalities for fire service, in one of its decisions, the California Commission referred to it as "Public Use Charge." The Commission apparently resorted to the enforcement of this charge upon municipalities to avoid increasing the rate for domestic and industrial service, as the water utility asked for a higher rate. The charge fixed by the California Commission for fire service in this particular case was evidently intended to be sufficient only to yield the water utility the additional gross revenue it was entitled to and had applied for. The charge fixed for fire service in this case resulted in being a trifle over 7 per cent of the water utility's gross revenue.

The California Commission has on numerous occasions made known to municipalities and apparently in no uncertain terms, its attitude regarding the fixing of a charge against municipalities for fire service.

This Commission stated in one of its decisions,

"... these municipalities must expect that at the appropriate time, when they are in a position to pay these increased charges, the rates of (name of the utility) will be revised with a view to lifting this burden from general consumers and placing it where it belongs, upon the municipalities."

So far reference has been made only to what privately owned water works have accomplished in fixing a charge for providing fire service facilities for municipalities (the general public). Why cannot city owned water works fix a charge for similar service and relieve the general consumer of the burden of the cost? Why should the water department in a city furnish free service to the fire department at the expense of the domestic and industrial water consumers?

All cities maintain a fire department equipped with modern fire fighting apparatus and the necessary man power to operate it. Are not the fire service facilities provided by the city owned water works for the exclusive use of the fire department and constructed of sufficient size to render an adequate service in case of fire? Would it not be fair then to relieve the general consumer of the burden of the cost and include it in the fire department budget and tax levy? This method if adopted would result in having those (the general public) who receive the benefit of fire protection pay for it.

In reviewing some of the audit reports of city owned water works there appears to be no set-up for such a charge. The city of Los Angeles does in its income statement show the value of some of the

water service the city receives free. The value of such free service is shown as part of gross operating revenue and is finally deducted from net income, making it merely a bookkeeping transaction.

### THE RATES OF THE EAST BAY MUNICIPAL UTILITY DISTRICT

It is thought that it will be of interest to refer to the charge made by one publicly owned water works in California, the East Bay Municipal Utility District, for providing fire service facilities to municipalities. The Utility District occupies rather a unique position regarding the fixing of rates and charges for the various classes of service it renders. Within its boundary line, on the continental side of San Francisco Bay, are nine municipalities. The District is not city owned. The municipalities are, in fact, the District's customers. Each municipality pays to the District a "readiness to serve" or "stand-by" service charge for providing facilities available for fire service. The amount of revenue derived from the charge for this service is approximately 7 percent of the total gross revenue of the District. The charge is much less than what it should be, considering the value of the service rendered.

What has been stated herein is intended merely as an exposition of what has been accomplished by the management of water works and rate making bodies, in arriving at the cost of and fixing a charge for, fire service furnished to municipalities. It is evident that much progress has been made. The trend is and as it rightfully should be, in water works operation, to relieve the ordinary user of water of all costs that are chargeable to other classes of service. Practically 90 percent of water works consumers are the home and industrial users, and the service rendered to this class is charged for according to the measured volume of water used. Consumers of this class then should not be required to pay any more than the cost of the particular kind of service they receive.

The fixing of a charge by water works for "readiness to serve" or "stand-by" service rendered the general public in municipalities for fire purposes will, if made effective, reduce considerably the present cost of service rendered to other classes of consumers.

# A NEW DISTRIBUTING RESERVOIR IN SYRACUSE[1]

By E. P. Stewart[2]

The City of Syracuse takes its water supply from Skaneateles Lake, which is located approximately 19 miles away in a southwesterly direction. The water is conducted to the city by gravity through two 30-inch conduits and 11 miles of a third conduit, 36 inches in diameter.

One of these conduits flows directly to a standpipe supplying the higher areas of the city, whereas the other two flow to Woodland Reservoir, which supplies the lower section of the city. As the high service consumption is about one-third of the capacity of the high service conduit, there is an over-flow from the standpipe which empties into the reservoir. Woodland Reservoir, for the last 35 years, has been the only distributing reservoir in the city, having a storage capacity of 121,000,000 gallons. It is 36 feet deep and has a surface area of approximately 14 acres. Its elevation is about 245 feet below that of the lake.

The storage in this reservoir was considered adequate for the needs of the city until 1927 when the total daily consumption had increased to 29,000,000 gallons per day. In the event of the failure of both conduits from the Lake, there would be but three or four days supply in the reservoir, even if restrictions were placed on the use of water during this period. At the end of two days several sections of the city would have little water pressure. As it was not impossible that such a failure of the conduits might occur, and as such an accident had occurred for a period of 24 or 48 hours, it was deemed advisable to construct another distributing and storage reservoir for the protection of the City's water supply.

Studies of at least a dozen sites were made, many of which were practical locations for a reservoir to meet our needs. From economic and strategic points of view, however, it was desirable to locate the reservoir at the western limits of the city. Two good locations were

[1]Presented before the New York Section meeting, September 23, 1931.

[2]Division Engineer, Department of Engineering, Division of Water, Syracuse, N. Y.

available in this section and for various reasons, a site was chosen on the top of a hill on the Knapp Farm. The proposed basin was, therefore, called Knapp Reservoir, but recently renamed Westcott Reservoir.

## WESTCOTT RESERVOIR

Westcott Reservoir is located approximately 1 mile north of the present conduit lines and about 1½ miles west of one of the main feeders from Woodland Reservoir to to the center of consumption of the city. There is a town road extending from the Reservoir to the Conduit Lines which was suitable in which to lay the inlet feeder, and an unimproved city street in which to lay the outlet feeder to the city.

The elevation and size of the hill was such that a reservoir of the same elevation and depth as Woodland and of approximately the same capacity could be economically constructed, the capacity being about 105,000,000 gallons.

The geological formation of this hill is commonly known in this section as a drumlin, having been deposited by glacial action. Preliminary borings indicated the soil to be mostly hard clay with a mixture of gravel.

Plans for this reservoir were prepared in the office of the City Engineer and work begun on the project in the fall of 1927, by John Young Contracting Company, a local contractor.

According to plans the central portion of the hill was excavated and an embankment formed around the perimeter of the hill with the material excavated, to such lines and grades that approximately one-half of the embankment is natural ground and the other half artificial. The slope of the embankment is one on two. The width of the embankment at the top is 15 feet. A berm 10 feet wide is located around the outside of the embankment at an elevation of 21 feet below the top. About one-half of the total excavation, was spoiled about the grounds for landscaping purposes. Work on the embankment and excavation of the bottom was not completed until the fall of 1929.

A sand-pocket was encountered in the northeast corner of the reservoir, but aside from this small area, all of the materials which were used in the embankment are of excellent quality for this work, varying from the hardest clay with a small mixture of gravel, to lighter clays with some sandy material. The latter however, has

sufficient clay to prevent seepage. In placing the embankment, the material was spread in layers not exceeding 12 inches in thickness and rolled uniformly with loaded trucks.

For the convenience of the contractor a gap was left in the northwest corner of the embankment until the completion of the job to provide entrance to the basin. In order that the compaction in this gap when filled, might be equivalent to that of the rest of the embankment which had been settling for over a year, special care was taken in selecting and placing the material, with the result that no appreciable difference in settlement is apparent.

On account of the sand-pocket found in the original ground in the northeastern corner of the basin, it was deemed advisable to fill the reservoir with water and test the imperviousness of the embankment prior to the placing of the lining. This was done in the Spring of 1930, a maximum depth of 10 feet of water being used. Borings were made in the embankment in the region of the sand-pocket, and in other parts of the embankment, to a depth below that of the bottom of the reservoir. Soon after the water had reached a depth of 10 feet, it was found that the water level in one of the borings opposite the sand pocket, had reached a level almost equivalent to that in the reservoir. Although there were no signs of seepage visible on the outside face of the embankment or the outside toe of slope, it was concluded that the inner slope of the embankment in this area should be faced with a blanket of impervious clay. This work was done in the summer of 1930 with the City's own forces. The embankment was excavated on the inner face to a depth of about three feet and also a trench was excavated at the toe of slope to a depth sufficient to secure impervious material. A good quality of clay available from previous excavation was then rolled into very thin layers into this trench and excavation on the slope.

In July, 1930, bids were taken on three different types of lining for the reservoir, namely, all gunite; gunite slopes and concrete slab bottom; and an all concrete slab lining. All three propositions called for facing the upper 20 feet of the inside slope with Onondaga Gray Limestone or granite surrounded by a 6-inch top curbing of the same material. A 6-inch layer of crushed stone was placed under the lining on the slopes. At the bottom of the crushed stone is a 6-inch open jointed drain tile which is connected to a drain outside of the reservoir. The purpose of the crushed stone lining is to conduct any seepage from the embankment or leakage from the reservoir to a drain where it can be observed at all times.

The contract for grading and lining the reservoir was awarded to I. M. Ludington's Sons, Incorporated, of Rochester, New York, for an all gunite lining, as this construction was lowest in price and as there was some doubt as to the practicability of pouring impervious concrete slabs on a one on two slope.

The guniting work was sublet by I. M. Ludington's Sons, Inc. to the Wertz Company, of Cleveland, Ohio.

It was decided to place expansion joints in the gunite lining at intervals of a 100 to 150 feet on the slopes and in squares of approximately the same dimensions on the bottom. These joints were made in the following manner.

*Joints in gunite lining and placement of gunite*

Concrete battens were poured below the subgrade of the lining and these battens coated with pitch to form a slip joint. The two slabs of gunite almost meet midway on the batten and a plastic material which is not materially affected by a change in temperature was placed in the space between them. A copper strip, about 10 inches wide with a small "v" formed in the center for expansion purposes, was incorporated in the gunite slabs so as to form a water stop in the joint.

The gunite was shot in two layers, the first layer being 1 inch in thickness and the latter 2½ inches in thickness. The reinforcing mesh was placed after the first layer of gunite was shot, thereby insuring its proper position in the slab. Core borings from cured slabs showed that for the most part a good bond was obtained between the layers of gunite. A better grade on the crushed stone was maintained by placing this mesh after a layer of gunite had been shot. A toe wall at the top of slope and an anchor wall about midway from the toe of slope to the top of the embankment were placed to resist any tendency of the embankment lining to slide or buckle. Onondaga limestone was chosen for the stone facing work on the upper portion of the lining, and the same material was used for the stone curbing. The 6-inch stone facing, known as Roman Rubble, was laid up with mortar upon a recess in the lining made to receive it. Not only does it present a pleasing appearance, but it will resist wave and frost action at the water line which is about half way up the stone facing when the reservoir is full.

*Leakage*

This work was completed and the reservoir filled for test in July 1931. Two small leaks at joints around structures were discovered before the reservoir was quite full. It was lowered again to one-half its depth and repairs made by means of a diver to one leak above water around the gate house foundation and to one leak below water around an outlet drain pipe. On filling the reservoir again and retesting, it is found that the total leakage, which is conducted out through a drain where it can be measured, is between 15,000 and 20,000 gallons per day. This is less than half of the allowable leakage specified in the contract. As the total area of the gunite lining is approximately 560,000 square feet the leakage is about 0.03 of a gallon per square foot of lining area. Measured on the level of the reservoir, this would amount to about 0.05 of an inch per day.

*Pipe connections*

As previously stated, Westcott Reservoir is located at a distance of about 1 mile north of the present conduits which feed Woodland Reservoir and on a town highway which crosses these conduits. The 42-inch feeder was, therefore, laid in this highway a few feet off the existing concrete pavement. This feeder is tapped into Conduit No. 1 and Conduit No. 2 by means of suitable crossover connections with valves. A gate house has been constructed at this point known as Andrews Gate House. This has become an important point in operating the system as the flows to the two reservoirs from the two existing conduits can be diverted in any manner wished by the manipulation of the valves which have been installed so as to give a very flexible control. Practically all valves are in duplicate, one valve of each set being hydraulically operated and the other hand operated.

For the present both reservoirs are fed from the low service Conduit No. 1 and provision has been made in Andrews Gate House for feeding either reservoir with Conduit No. 3 as well as Conduit No. 1, when the former shall have been completed from the lake to the city. As Conduit No. 2 is used to fill the high service standpipes, it will not be used for supplying Westcott Reservoir except during unusually high consumption.

The piping and valve arrangement at the new reservoir is simple, yet it is quite flexible. The 42-inch feeder enters the lower gate house and is reduced to 30 inches in order to allow the use of smaller

valves. The water passes through a 30-inch Ross Valve which serves the triple purpose of altitude valve, reducing valve, and hydraulic shut-off valve. A 30-inch hand operated and a 30 inch hydraulic gate valve are located in the same line and on either side of the Ross Valve which then increases to 42 inches and passes through a concrete tunnel or pipe gallery through the embankment to the dry well of the upper fate house. In this stretch of pipe is located the segmental orifice which actuates the meter which records the flow. The 42-inch feeder continues through the dry well and out into the reservoir, a distance of approximately 700 feet. On entering the reservoir proper it is supported on concrete piers just about the bottom. On the end of this feeder is located an aerator structure in the bottom of which the pipe again reduces to 30 inches in size. A riser to the spray nozzles, 20 inches in diameter, is located just ahead of another 30-inch Ross Valve which can be controlled from the lower gate house. Closing this valve forces all of the water up the 20-inch riser through the aerator nozzles and upon opening the valve the water passes through it and into the reservoir directly through a 30-inch flap valve. The Ross Valve is provided with a suitable pilot for maintaining any desired pressure on the aerator.

The outlet from the reservoir to the city is at the south end at the same point where the 42-inch feeder enters. However, as the inlet feeder discharges near the north end of the reservoir adequate circulation in the reservoir is maintained.

The outlet is a 4 by 4 feet concrete tunnel extending through the embankment into the wet well of the upper gate house where the water passes through a $\frac{1}{4}$-inch screen. Leading out of this well and through another concrete pipe gallery extending through the embankment is a 48-inch cast iron conduit which passes through the lower gate house and thence to the city. On this feeder is located a 48-inch sluice gate in the upper gate house and a 48-inch hydraulically operated gate valve in the lower gate house. Suitable cross-connections and valves have been installed in the lower gate house for bypassing the reservoir entirely or partially during periods of unusually high flows.

Another segmental orifice is located in this feeder by means of which the flow is measured.

This outlet feeder continues 48 inches in size to the city, a distance of about $1\frac{1}{2}$ miles, passing along an unpaved city street, through a city park, the grounds of a State Institution and a block of paved

street to connect with an existing 30-inch distribution feeder. At this point is located another 30-inch special Ross Valve which can be used as a hydraulic shut-off valve. It is also equipped with pilots which will automatically shut the valve for reduction in pressure on the reservoir side. It is felt that this feature is a factor of safety against damage from a possible break in this large supply main. In case of such a break it is necessary to operate but one hydraulic valve at the reservoir and the valve at the other end of the feeder will automatically close.

There are two gate houses at Westcott Reservoir, one at the top and one at the bottom of the embankment. They are circular in shape and are constructed of white artificial stone on the exterior and lined with yellow tile. The roofs are of red tile.

Inasmuch as Westcott Reservoir under normal conditions will be supplied from the same conduit as Woodland Reservoir and as their outlet feeders are connected together it has not been deemed necessary to maintain three shifts of operators. Any changes in flow at Westcott can be noted at once at Woodland as Woodland Reservoir is completely equipped with flow recorders, located in the operator's office. The plan is to construct an operator's residence at the reservoir with but one man on duty practically all of the time. The operator's house has not been constructed as yet, but it is expected that it will be during the winter or the early spring.

A 5-foot chain link fence has been constructed around the top of the embankment at a distance of 5 feet from the curb.

The grounds are now being landscaped by the Park Department of the City, using plans developed in the office of the City Engineer. As the reservoir is located on a hill, the slopes are rather steep and for this reason the entire grounds are being sodded rather than seeded.

This reservoir has been in operation for about one month and has proven to be very satisfactory. Inasmuch as Westcott Reservoir is located a little closer to Andrews Gate House than is Woodland Reservoir and as its feeder capacity from Andrews Gate House is larger than that to Woodland, there is a tendency for the water in the conduit to pass to Westcott Reservoir. However, this is readily diverted to Woodland Reservoir by throttling the aerator valve and raising the head. In this manner a perfect balance can readily be obtained of the two reservoirs.

# PURIFICATION OF WATER SUPPLIES[1]

By Richard G. Tyler[2]

The water supplies west of the Cascades are principally soft, low in alkalinity and with a pH ranging from slightly acid at high river stages, to slightly alkaline at low stages. The color of these waters varies with the rainfall and the amount of tributary swampy area. The turbidities are not high, as a rule, and are usually coarse except where glacial flour is present. After heavy rainfall particularly, turbidities may be present in objectionable amounts.

East of the Cascades the waters the waters of the Pacific Northwest are much more alkaline, except those coming from the mountains themselves. They have a greater range of turbidities, and carry considerably more mineral matter. Though the purification of these waters is more necessary, the problem is a simpler one, since the water is more easily coagulated. This paper, therefore, will be limited to a discussion of the problems involved in the purification of the softer waters of the western slope. These are more like the waters of New England than of any other section of the United States, but are less colored and have higher alkalinities.

The average water characteristics for a number of streams in Washington, with one analysis from Oregon, are shown in table 1. From this table the general character of the water on both sides of the Cascades can be seen.

None of these waters are very hard, that of the first six averaging about 27 p.p.m. as $CaCO_3$, while the other six Washington streams average about 55 p.p.m. A close approximation of the total hardness can be obtained from the formula

$$\text{Total hardness} = 2.5\ \text{Ca} + 4.0\ \text{Mg}.[3]$$

The only common requirement of all of these waters would be the necessity for chlorination. While some are relatively free from sew-

[1] Presented before the Pacific-Northwest Section meeting, May 15, 1931.

[2] Dean, College of Engineering, University of Washington, Spokane, Wash.

[3] Total hardness as $CaCO_3$ = 2.497 Ca + 4.115 Mg. (Iron not important in amounts present.)

TABLE 1

*Typical analyses of Washington waters*

*U. S. G. S. Water Supply Paper No. 339, 1914*

| 1910–11 | TURBIDITY | SUSPENDED SOLIDS | COEFFICIENT OF FINENESS | $SiO_2$ | Fe | Ca | Mg | Na AND K | $CO_3$ | $HCO_3$ | $SO_4$ | $NO_3$ | Cl | DISPERSION SOLIDS |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1. Skagit River at Sedro Wooley... | 12 | 19 | 1.31 | 9.4 | 0.08 | 7.9 | 1.7 | 3.6 | 0 | 29 | 8.2 | 0.24 | 0.9 | 48 |
| 2. Wood Creek near Everett..... | 10 | 13 | 1.15 | 25.0 | 0.01 | 8.6 | 4.5 | 7.6 | 0 | 51 | 7.8 | 0.31 | 2.9 | 86 |
| 3. Cedar River at Ravensdale..... | 5 | 4.2 | 0.74 | 13.0 | 0.02 | 6.7 | 1.4 | 3.6 | 0 | 28 | 5.7 | 0.20 | 1.2 | 49 |
| 4. Green River at Hot Springs..... | 6 | 7.0 | 1.10 | 17.0 | 0.04 | 6.0 | 1.3 | 5.6 | 0 | 28 | 5.9 | 0.13 | 1.3 | 55 |
| 5. Chehalis River at Centralia..... | 17 | 18 | 1.01 | 16.0 | 0.08 | 7.1 | 1.9 | 6.5 | 0 | 31 | 6.4 | 0.10 | 5.2 | 63 |
| 6. Wynoochee River near Montesana | 2 | 1.6 | 0.93 | 11.0 | 0.01 | 8.2 | 2.2 | 4.8 | 0 | 36 | 5.7 | 0.29 | 2.1 | 53 |
| 7. Spokane River at Spokane..... | 5 | 4.1 | 0.79 | 11.0 | 0.02 | 11.0 | 3.6 | 5.3 | 0 | 48 | 9.2 | 0.23 | 0.6 | 63 |
| 8. Okanogan River at Okanogan... | 25 | 24 | 1.21 | 14.0 | 0.02 | 21.0 | 4.6 | 8.5 | 0 | 81 | 16.0 | 0.28 | 0.8 | 110 |
| 9. Wenatchee River at Cashmere... | 9 | 7 | 0.76 | 12.0 | 0.03 | 5.5 | 2.3 | 4.2 | 0 | 28 | 7.4 | 0.31 | 1.0 | 46 |
| 10. Yakima River at Prosser..... | 18 | 17 | 0.94 | 19.0 | 0.10 | 16.0 | 6.1 | 14.0 | 0 | 80 | 21.0 | 0.34 | 5.2 | 123 |
| 11. Snake River at Burbank..... | 42 | 52 | 1.76 | 19.0 | 0.05 | 19.0 | 5.6 | 14.0 | 0 | 83 | 21.0 | 0.53 | 8.1 | 131 |
| 12. Columbia River at Pasco..... | 9 | 6.7 | 0.70 | 7.7 | 0.04 | 18.0 | 4.5 | 6.0 | 0 | 73 | 11.0 | 0.14 | 0.7 | 83 |
| 13. Bull River at Head Works, Oregon..... | Trace | 7.1 | | 9.0 | 0.02 | 25.0 | 0.12 | 4.6 | 0 | 11.7 | 3.2 | 0.31 | 2.0 | 26.2 |

**Results in parts per million.**

age contamination, the occasional hunter, fisherman or timber cruiser is a sufficient potential danger to make chlorination a wise precaution to take in all cases. I am aware that cities frequently boast that it is unnecessary to chlorinate their water supplies, and feel that postponing chlorination as long as possible is an advantage. This view is based upon a lack of appreciation of the factors involved and is not to be encouraged. Positive assurance that it is impossible for pathogenic bacteria to reach the consumer, even though present in the raw water, is of more value than the negative claim that there are no bacteria in the water. The cost of chlorination is so slight and the troubles which have sometimes occurred are so needless with our present state of knowledge, that there is no adequate justification for omitting the chlorination of any public water supply.

While the turbidities of these streams are not high, if the water is consistently turbid to the extent of 10 p.p.m. or more, either long time storage or filtration should be resorted to. Where natural lakes are available and the water can be taken directly from them, storage may furnish adequate purification so that filtration is unnecessary. Storage, however, brings its own special problems of tastes and odors from algae and other microörganisms.

The filtration of soft waters is more difficult of accomplishment than that of water with higher alkalinities because the floc forms more slowly and is more fragile. Longer sedimentation periods are necessary to allow time for satisfactory flocculation and for the majority of the floc to settle out so that it will not be carried over onto the filter. At Cambridge, Mass., where a soft water is coagulated with alum, sodium aluminate and soda ash, a sedimentation basin of 2½ hours normal detention period is too small. Providence, R. I goes to the other extreme, with a detention period of 4 or 5 days, and while this is very effective in saving chemicals and in securing long filter runs, only where similar topographic conditions make such storage relatively inexpensive, would such abnormal detention periods be justified.

The fragile floc passes down into the filter more easily and breaks through more quickly, thus reducing the length of filter run and cutting down the plant efficiency. A certain amount of turbidity adds to the toughness of the floc and furnishes nuclei about which it can form. Just as a fine sand or stone dust filler stiffens a bituminous street surface, so suspended solids toughen the particles of aluminum

hydrate. As the large, tough floc is held on top of the sand more readily than small, fragile particles, a coarser filter sand can be used with such waters than with soft. Referring again to the Cambridge plant with which I am more familiar, the best effective size of the sand was found to be about 0.31 or 0.32 mm., while at St. Louis 0.5 mm. is satisfactory.

Soft water, therefore, produces poorer floc, which requires longer sedimentation, finer filter sand, and may cause shorter filter runs. Lime or soda-ash is usually added ahead of the alum to give sufficient alkalinity for a complete reaction by the alum. Sometimes, however, this has not helped coagulation, but has, on the other hand, peptized or further broken up the color or turbidity so that these passed through the filter. In such cases it has been found more satisfactory to split the lime or soda-ash dose, applying part of it before and part after filtration, or in extreme cases, applying all of the alkalinity as the water enters the clear well.

There is a matter connected with coagulation of water that is frequently overlooked at the smaller plants, or at those where the plant is not in charge of a skilled operator. This has to do with the desirability of adjusting the pH to the value at which the particular water coagulates best. While this point varies with different waters, it usually falls between 5.5 and 6.5. That is, the water flocculates best when slightly acid. This being the case, the addition of alkalinity before adding the alum may give too high a pH for most successful coagulation and may, in fact, cause further dispersion of the color and turbidity, as above stated. The maximum solubility of tannin, which may be taken as an example of the material producing color in these waters, occurs at a pH of about 8.2 to 9.2, while it precipitates best at a pH under 4.6. It can be seen, therefore, why it may be advantageous to add the alkalinity after the water has been coagulated and passed through the filter. Enough alkalinity should be added in such cases to neutralize the $CO_2$ present and to leave a residual alkalinity of at least 10 p.p.m. as a guarantee against corrosiveness. Since pH plays such an important part in the coagulation process, it is extremely important that the plant operator understand its bearing upon his problems and the methods employed in adjusting pH to any desired value. In this connection it should be noted that lake water that has stood for some time is normally more easily coagulated than river water which has come more recently from snow or rainfall and in which the dissolved chemicals picked up en route have not reached a

state of equilibrium. At the Cambridge plant, for example, it was found that during the spring when the water was particularly hard to coagulate, the situation could be considerably improved by mixing this *new* water with that from Fresh Pond adjacent to the filter plant, where the water had stood for some time. In this case the supply had received some storage in collecting reservoirs, but the longer storage in Fresh Pond had apparently permitted a partial agglomeration of the finer suspended solids and a more complete equilibrium between the various suspended and dissolved materials in the water.

One of the difficulties encountered in treating lake water which has been noted at some plants is the clogging of the filters by algae which find more favorable growing conditions in lakes than in rivers. These algae may be particularly troublesome at certain seasons of the year, namely at the spring and fall turn-over periods, and the shorter filter runs which they cause is due to the clogging of the filter sand, either by the algae themselves or by gummy by-products of their metabolism. When such conditions ensue the shorter filter runs which they produce may considerably reduce the efficiency of the plant. In such cases it is sometimes necessary to wash the filter at lower head losses than under normal operating conditions, as otherwise some of the suspended materials will pass through the filter. It is desirable to destroy the algae before they reach the plant, if this is possible, and the better practice would be to treat that part of the lake adjacent to the intake with copper sulphate so that the dead organisms may have time to settle out before reaching the filter plant. One advantage of placing this treatment sufficiently ahead of the filters is that if algae have not settled out but have reached the plant, taste producing organisms may cause more than their usual amount of trouble when the water is chlorinated. If, for example, as in the case of the Seattle water supply, copper sulphate is applied at Landsburg to kill the algae, and chlorination takes place in the conduits near Lake Youngs on the way to the city, it is more difficult to prevent the production of odors and tastes, as the chlorine appears to fix or accentuate the taste from the dead algae. However, with the ammonia chlorine process which is used in this particular instance, it is possible to handle the situation and adequately remove these tastes.

In the coagulation of water we have been accustomed to using alum, lime, soda ash, and perhaps ferrous sulphate or "copperas." The iron and lime treatment is not ordinarily suitable for soft waters,

as it requires a high pH, say 9.0, for satisfactory coagulation and the color which is frequently present in soft waters reaches its point of maximum dispersion at about this pH. While alum is widely used with such waters, the writer believes that sodium aluminate or ferric chloride (chlorinated copperas) may be even more effective as coagulants. The latter is effective over a wide range of pH, which is advantageous in treating a water of such widely varying characteristics. Experiments with this coagulant show that it is particularly effective in removing turbidity. It would be interesting to experiment with ferric chloride in the treatment of some of these waters. It is easily produced at the plant by treating ferrous sulphate with chlorine in the ratio of about 8 to 1 by weight.

Sodium aluminate is ordinarily used in conjunction with alum and usually reduces the amount of alum required. It effects a saving also in soda ash where the latter is used.

Such municipalities as I have come into contact with in this section of the country have very meager data concerning the character of their water supply and the way in which this varies throughout the year or over longer periods of time. Occasions will arise, as for example in the case of law suits involving the municipal water department, where the department will wish that it had kept more complete data and records on the quantity and quality of its supply. Data on temperature, taste, odors and chemical and bacteriological content of the water over long periods of time are sometimes very useful. Such data are of service in planning extensions or enlargements of the present supply or in measuring deterioration or improvement of its source. There are many other reasons also why such data, regularly kept, are desirable.

A brief comment should probably be made concerning the use of activated carbons for the removal of objectionable tastes and odors from water supplies. While this work is still in an experimental stage, investigations by Baylis and others, together with some full sized installations, have indicated the high efficiency of these materials in taste removal. The method is simple and effective and it would be interesting to see some of the municipalities in the Northwest experimenting with this material in the handling of their tases and odor difficulties.

In conclusion, the waters west of the Cascades are of such a nature that while they can be satisfactorily treated by filtration with coagulation, their chemical characteristics require a moderate amount of

skill for satisfactory operation of filter plants. The problems which arise, however, are not very serious and any operation should be able, with some experience and advice, to meet the situations arising at this particular plant due to variations in the characteristics of the water treated.

# HYDROLOGIC RESOURCES AND PHENOMENA OF OREGON[1]

By H. S. Rogers[2] and F. Merryfield[3]

Hydrological studies are made for their value in the general description of physiographical phenomena and particularly for their use in disclosing the principles of these phenomena and supplying the data necessary, first, for the study of projects dependent upon water resources, and second, for the regulation of operating conditions of constructed projects. The most reliable data upon which projects may be developed are, of course, those collected through direct observation over a period of time sufficient to establish their reliability. It is, however, frequently necessary to proceed with the construction and development of projects upon which the quantity of observed and measured data is very limited and in which it is, therefore, desirable to utilize analogous data as a check and guide.

The fundamental data necessary for the study of water supply, sanitary, power, irrigation, drainage, flood control, and navigation projects include (1) the flow of streams in terms of continuous rates, maximum flows, and minimum flows, and gauge height, (2) the movement of ground water as related to seepage and return, (3) the amount of rainfall, snow cover and evaporation, and (4) stream profiles and velocities and topography. All of these data are collected at the present time by various governmental agencies. That relating to flow of streams, ground water movement, the stream profiles and topography is collected by the U. S. Geological Survey and the State Engineer's Office. That related to precipitation and to gauge height is collected by the U. S. Weather Bureau. Certain miscellaneous data are collected by the U. S. Engineer Corps, U. S. Reclamation Service, various power companies, and some of the larger cities. The extent of data necessary in the study of any project depends, of

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course, upon the magnitude and significance of the proposed investment and works.

## PRINCIPLES OF HYDROLOGICAL PHENOMENA

While projects involving large expenditure, such as the Lewis River development of the Northwestern Electric Company, will always require years of direct observation and statistical study, many smaller projects must proceed upon limited data. An understanding of the principles underlying hydrological phenomena in the region in which they are built is a necessary precaution in the application of indirect data and in judgments upon project details. The hydrological phenomena of Oregon may to some extent be taken as characteristic of the general northwest territory. They may be studied under the categories of climate, precipitation, and runoff.

The major elements controlling the climate of Oregon are the planetary circulation of the atmosphere and the geographical position of the state with respect to the Pacific Ocean. The rotation of the earth and the temperature gradient between the equator and the north pole create the more or less permanent Southwesterly winds which blow from the Pacific Ocean over the Western slopes of the Cascade and Coast mountains. Between the fall and spring equinoxes these winds sweeping from the ocean areas are laden with moisture which they precipitate as they are deflected upward by the Coast and Cascade ranges, across their path. They are the major forces responsible for the winter climate of Western Oregon. Tempered by the vast Pacific area, they give us our moderate and uniform climate. At times, but infrequently, a reversal of atmospheric flow brings cold winter winds from the interior land areas and causes a general precipitation of snow. The shifting of this belt of wind movements toward the North during the summer period and the increase in land temperatures give us the control conditions of the long continued months of relative drouth.

Three fundamental causes underlie precipitation over land areas, (1) the direction of prevailing winds, (2) the source from whence they come and its influence upon their moisture content, and (3) the dynamic cooling of air resulting in precipitation. This dynamic cooling may be produced by the deflection of winds upward as they encounter mountain ranges in their path, by cyclonic movements resulting in atmospheric vortices, and by upward convections due to the heating of air over land areas. The effect of vertical deflection

of winds by ranges normal to their paths is to produce an increase in precipitation in proportion to the altitude of the terrain over which they flow. The rainfall map of Oregon, therefore, looks very much like a map showing the topographical contours.

The precipitation along the Coast of Oregon is in a general sense heavier than inland, but increases to the crest of the Coast range and decreases throughout the inland valleys paralleling the range. It again increases as the winds sweep over the Cascades where they are almost denuded of their moisture and is found to be uniformly light throughout the high plateaus in the eastern part of the State. The rainfall along the Coast is quite uniform and that along the line between The Dalles and Klamath Falls East of the Cascades is also very uniform. The latter phenomenon shows a departure from the relationship between rainfall and altitude because the winds blow across instead of along the North and South line and are denuded of their moisture before they reach the lateral areas.

Because of the uniformity of winter wind movements the rainfall conditions in the state are perhaps more uniform than in any other state in the Union. The maximum intensitives of precipitation are much lower than in the Middle West and East; the departures from the average annual rainfull are also much lower. The distribution of annual rainfall is the reverse of that found in the Middle West with the valley of the curve in the summer months and the peaks in the winter. These phenomena are similar in both Western and Eastern Oregon although, of course, the quantitative difference between winter and summer rainfall is much larger along the Coast than East of the Cascades.

The variations of runoff in Western Oregon closely parallel the precipitation except as it is delayed and distributed by ground and snow storage. This ground storage in the Coast range with its cover of timber and brush is high as compared with middle western areas but is not so great as the snow storage along the high levels of the Cascades. The snow storage supports a very uniform and stable flow of streams originating along the western slopes of the range during the dry months of the year. The effect of snow, however, is not of a nature to obliterate the flood flows paralleling the heavy storms of the winter.

The topography of the Coast range and of the Western Cascade areas is such as to produce rapid runoff and high flood peaks under storm conditions. Although the effect of geological formations has

not been quantitatively determined the lava flows of central Oregon exercise a marked influence upon the discharge of the Deschutes and Klamath Rivers. The general conditions throughout the state are such as to create a greater variety in runoff than perhaps in any other section. This is in contradistinction to the unusual uniformity of rainfall. The Rogue River of Western Oregon is one of the flashiest of streams. Its peaks rise suddenly and are as suddenly depressed. The Deschutes of central Oregon, on the other hand, is one of the most uniform streams. It is fed by tremendous springs which discharge large quantities of water in the summer time in the heart of a country parched by heat and drouth and its highest flow is not more than 300 percent of its mean. The flood peaks of the Willamette occur in December and January while the flood peaks of the Columbia into which it discharges occur in June. These conditions unite in producing hydrological phenomena that can be extended and utilized by analogy only with a great deal of care and discretion growing out of critical understanding. Certain data should, however, be of considerable value for analogous studies in adjacent and similar areas lying on the West of the Cascades and along the coast regions.

## METHODS OF PRELIMINARY PROJECT STUDY

There is, of course, no other hydrological method of study comparable to that of direct measurements made specifically for project purposes. The most satisfactory data upon stream control are the continuous hydrographs constructed from daily observed rates. These hydrographs can be satisfactorily used in terms of average weekly or monthly flow where even very limited regulations are available. The choice between daily, weekly, or monthly mean flows in their application to project studies should be conditioned by the extent of regulations anticipated or existing. These hydrographs showing rates of flow are frequently summated into mass diagrams for convenience of study and presentation.

The amount of storage necessary may be determined either from hydrographs or mass diagrams. Where storage extends over a period longer than one season of low flow, the mass diagram is more convenient for comparison and study than the continuous hydrograph. Evaporation corrections have been used in irrigation studies, but are subject to much error in the present state of quantitative determination and prediction. When coupled with seepage estimates the two should be used with a wide margin or safety. The large and

expensive developments of Oregon have been made only after the expenditure of considerable sums in direct hydrological measurements.

To compare hydrological phenomena will always be a challenge to those scientifically inclined. Such comparisons will always be valuable to supplement and check direct observations and on many public projects that are sometimes forced ahead without long and continuous direct measurements. They are also valuable in determining the feasibility of expenditures for direct measurements.

Comparative hydrographs are of value and can be used in the study of similar watershed or by extension from one fork or one area in a particular watershed to a larger area of the same. Few Studies of this type have been made in Oregon. One upon Bear Creek in Jackson county made by F. E. Dillard and checked by State Engineer C. E. Stricklin proved very satisfactory. It is, nevertheless, one of the few, if not the only one which has been made. There are two fundamental bases for the study of comparative hydrographs, one, the average unit of runoff per square mile, and the other the coefficient of runoff based upon observed rainfall. These like hydrographs may be studied in terms of average annual runoff or by means of mass diagrams. The amount of runoff that does not conform exactly to calendar dates may be conveniently studied over seasonal periods by mass diagrams. The use of coefficients of runoff over any appreciable area requires the construction of maps showing the annual isohyetal lines. The method is, there, limited in its application to Oregon problems because of the great distances between rainfall stations and the consequent lack of fundamental rainfall data. The data which may be used more satisfactorily for comparative purposes are, therefore, the maximum, minimum, and average flow per square mile of watershed. An expression for determining the maximum runoff or flood flows of western streams was determined for the Federal Land Bank by J. T. Whistler for the purpose of judging the safety of spillway capacities. It is difficult to imagine accurate results from the application of such a formula to areas differing to the extent found between western and eastern Oregon.

The flow duration curve should be of some value in studying Oregon streams. A flow duration curve differs from a hydrograph in that upon it flows are segregated in their increasing or diminishing order rather than indicated in the order of continuity of time. Such curves are valuable in determining the flow characteristics of a stream and in comparing these with others, as well as for determining the

period of time over which records must be studied accurately to determine characteristics. A study of several of Oregon's streams including the Willamette at Albany, the Columbia at The Dalles, the John Day at McDonald indicates that a ten year period of study will give these characteristics, accurately except for the extreme flood flows and extreme drouths. Since it is necessary, however, in project development to determine the characteristics of annual or seasonal flow, the flow duration curve to be used for such purposes must be the minimum envelop of yearly curves. Flow duration curves have been used for comparatively studying streams in North Carolina and may be of value in the study of the Pacific Coast streams. The value of comparative hydrographs is, of course, determined by the conditions under which the fundamental data were collected and must be checked by reference to the amount of rainfall during the period. Rainfall records are available over long periods of time and indicate the departure from average or specific conditions. The growth rings of trees have been studied recently for determining the long time fluctuations in rainfall. They appear to be of intellectual interest and to show general fluctuations, but offer little accurate quantitative measurement for determining rainfall variations.

## METHODS OF FORECASTING

Two methods of operating forecasts have been used with considerable success: one, based upon the correlations between summer rainfall and stream flows has been used in North Carolina in controlling the operation of hydroelectric plants during drouth periods, and, the other, based upon the correlation between snow cover and summer runoff, has been used with satisfactory accuracy in the West. Forecasts in North Carolina were developed in the State Department of Conservation and Development and perfected by a determination of the coefficients of relationship and the time lag between rainfall and run off.

The studies of forecasting from snow cover were developed by J. E. Church of the University of Nevada who perfected the technic of sampling and the determination of water content. The process is simply that of establishing the water content of snow cover over the area and determining its amount as it might be determined from average precipitation or from isohyetal lines. Snow surveys have been made in Oregon for the past three years upon areas in the Malheur, Powder, Grande Ronde, Walla Walla, Umatilla, John Day,

White, Rogue, and Klamath river drainage areas and in several of the larger lake drainage areas by the state engineer's office. In these surveys the water content was determined about the first of January and again about the first of April. The results indicate the possibility of rather accurate forecasts and control upon the use of irrigation water.

Other methods of predicting probable rainfall have various degrees of dependability. The annual rainfall of Corvallis plotted in percent of the mean against month of the year indicates the probability of predicting at least a continuation of dryness or wetness during the latter months of the water year. Professor A. J. Henry of California has pointed out that the "precipitation in California is great or small according as to whether areas of low pressure enter the continent and move eastward in low or high latitudes" and L. E. Blochman has presented a theory of forecasting wet seasons by the entrance of low-pressure areas across the coast line in September or October. All satisfactory methods of forecasting apply particularly to periods of minimum flow or seasons of drouth, because minimum flows extend over much longer periods than flood peaks and dry seasons last longer than storms of maximum precipitation.

## REGULATION OF WATER RESOURCES

The water resources of Oregon are controlled by various state and federal statutes. The Federal Power Commission exercises jurisdiction over all power projects involving the use of government land or navigable streams under the provision of the federal water power act. The Power Commission has withdrawn at power sites in Oregon a total of 142,672 acres.

The state laws authorize the state engineer to adjudicate the water rights upon the streams of Oregon. "Up to the present time findings and orders of determination have been made adjudicating the rights upon sixty-one streams, including irrigation rights for well over a million acres of land." The determinations are subject to review by the court and have been generally upheld with few if any modifications. As soon as the water rights on a certain stream have been determined by the state engineer a water district is created in accordance with the law. The boundary of any existing water district is extended so as to include the stream system. The regulation of the quantity of water diverted from a stream so organized is then under the direction of the state engineer.

The state laws also provide for the diversion and storage of water from open streams and a recently enacted law applies to the development of water power. Certain clauses in the health, fish, and game laws of the state also apply to the control of stream conditions. The ground waters of eastern Oregon are controlled by legal procedures for filing upon them. The law relating to them will probably undergo revision before it is applicable to conditions throughout the entire state. A new pamphlet upon the Oregon water laws will be available at the state engineer's office in a few days.

The conditions existing in Oregon will permit the development of unique laws for controlling stream pollution during low water seasons. Since the summer flow of Oregon streams is so uniform it would be possible to grant permits for the discharge into them of certain limited quantities of wastes, depending upon the amount of dissolved oxygen which might be absorbed in a stream as cities and irrigation districts now file upon the water and permits might be granted for a limited period of time so that adjustments to changing conditions would be provided.

## CONCLUSIONS

The study of Oregon's water resources and hydrological phenomena has been carried forward continuously since 1905 through the co-operation of the U. S. Geological Survey and the State Engineer's office. Records of daily flows of streams throughout the state, including those of all the more important gauging stations, have been or will be published in the reports of the Geological Survey. The U. S. Geological Survey is authorized to co-operate with the state and municipal bodies including cities and counties, with individuals or companies holding permits or licenses of the federal power commission or otherwise engaged in investigating or developing the water resources of the streams in the state, or to co-operate with other federal bureaus. Most of the larger power companies have collected runoff data by co-operation with the Geological Survey. The Thirteenth (1930) Biennial Report of the State Engineer presents a map showing the locations of stream gauging stations in the state and gives a list of those in operation.

The California-Oregon Power Company has conducted some independent studies upon the runoff in the Klamath River at Upper Klamath Lake and is working upon methods of forecasting seasonal stream flow. Interesting data upon this work are presented in the

May 1929 issue of the *Engineering News-Record* by John F. Partridge. The City of Portland has independently collected data upon the Bull Run watershed over a long period of years. Other cities may be gathering data which should be made available by transmitting them to the state engineer or the district engineer of the U. S. Geological Survey, Water Resources branch.

The natural hydrological characteristics of the streams of many eastern states are obscured by the regulation resulting from impounding reservoirs and the opportunity to gather fundamental data for comparative uses has, therefore, been lost or complicated. There is immediate need for full co-operation in the collection of all possible data upon Oregon streams so that these fundamental characteristics may be described by studies extending over periods of sufficient length. The water superintendents have an opportunity in many cases to gather data concerning average, maximum, and minimum flows and these data could be made available to all interested parties through the U. S. Geological Survey. The Northwest Section of American Water Works Association should organize committees for gathering or promoting the collection of these data on the characteristic areas of the Pacific Northwest, such as that west of the Cascades, that along the coast, and that between the Coast and the Cascade Ranges.

# COLOR REMOVAL IN PACIFIC COAST WATER SUPPLIES[1]

By Paul F. Bovard[2] and Kenneth Shibley[3]

Water may be regarded as one of the most potent solvents which the chemist has under investigation. This property is so far reaching in its effect that engineering studies, scientific investigations and treatises on industrial development are, to a more or less extent, dealing with it. The Iron and Steel Industry exert their most serious efforts to inhibit this destructive force upon metallic iron. Other metallurgical researches are directed against the inevitable force which is, apparently, insatiable. The very structures upon which man depends for shelter and the articles, the manufacture of which combined make up the great industrial centers, are largely dependent either on the solvent property of water or our ability to devise methods to resist it.

It would, therefore, seem unreasonable to form any concept regarding natural water supplies which would disregard an inherent chemical characteristic so vital. All of the elements so far discovered are soluble, to a limited degree, in water. The extent of chemical solubility is of no direct importance as this degree of solubility is affected by temperature, pressure and further concentration of chemical combinations by prior solution contacts. The waterworks engineer may well be apprehensive, at times, over the selection of a domestic water supply and the industrial engineer careful in his analysis of the quality of water. The water passing through any pipe line exerts a solvent action on the iron and, even if the tendency is retarded, the previous contacts with mineral or vegetable bodies may render the water useless in its natural state. Manifestly, the corrosive action of water is due to its property of dissolving carbon dioxide and oxygen, setting up an unstable electrochemical energy partially neutralized by the positive electrode of iron. Other chem-

[1] Presented before the Pacific Northwest Section meeting, May 15, 1931.

[2] California Filter Company, San Francisco, Calif.

[3] California Filter Company, Seattle, Wash.

ical combinations are found in solution, causing hardness. Calcium and magnesium, associated with negative ions of sulfate, chloride, and nitrate, form a basis for serious industrial, as well as domestic, difficulties. In every case the activity of chemical solution is met with by known scientific reactions. It is, indeed, reasonable to expect that a reagent as vigorously active, chemically, would be selective in its attack on organic as on inorganic matter.

Natural streams contact in surface area, in some cases, more organic or vegetable matter than inorganic or mineral. Indeed, the streams tributary to our rivers in the Northwest may be all thus classified. Vegetation varies in its degree of solubility to the same extent as mineral combinations; depending on its composition. As vegetation is exceedingly active chemically, it is to be expected that its presence in water supplies would be given appropriate study. The fact that organic concentrations have not been seriously regarded, perhaps, is due to the fact that until about the year 1910 water supply engineers and industrial interests had the vast resources from which to select. If the most available surface supply was too highly colored for asthetic taste, wells or other streams could be tapped or the site changed to suit the emergency. The time seems to have arrived, however, when the quest for water will take precedence over the mere selection of one naturally suitable and meeting certain fastidious requirements.

## ORIGIN OF COLOR

Organic matter in solution is prominently noticeable by either taste, odor or color. The water may have any one or all of these qualities. Taste and odor due to organic decomposition and solution may be eliminated, in many cases, by chemical reagents. Color is not eliminated by the same treatment; being a complex derivative of totally different properties.

Where the water contacts fresh or newly grown vegetation, color solubility is at the lowest potential. The unoxidized or growing vegetation yields some organic oils, but color is not thereby visible. The oxidized vegetable decomposition which takes form after the period of bloom may be classified as alkaloids, organic anhydrides and acids. These are soluble to a marked degree in water. Chemical analysis reveals no relationship of importance between the amount of dissolved organic matter and the carbon dioxide content in natural streams. Neither is the color intensity necessarily a function of the

amount of dissolved organic matter or the oxygen demand. Apparently color is somewhat unrelated and is dependent on the class of color colloid present; much the same as the same quantity of black or green tea in the same concentrations yield marked differences in color intensity.

## COASTAL WATERS

Studies on Coastal water resources reveal interesting variations in color content. The large rivers have been generally regarded as free from noticeable color. Smaller streams draining through dense vegetation have received sufficient attention to be questionable as potential sources for either industrial or domestic supplies. However, the seasonal fluctuation in color in the very large streams indicated a very remarkable merging of colored tributaries. Color tests are assumed to refer to the United States Geological Survey Standards in parts per million and represent color intensities varying from a very slight amber to a deep brown. Color cannot be detected by the naked eye when the concentration is less than 5 p.p.m. When the color exceeds 70 p.p.m., aliquot parts are taken for accurate colorimetric readings.

The Columbia River has the usual varied geological features within its drainage area. A stream of its remarkable magnitude and length would traverse areas of mineral and organic contacts sufficient to effect its chemical characteristics with seasonal regularity. Much of the Columbia's drainage area is in Eastern Washington where vegetation is limited, thence through the Cascade Gorge where a sudden change of drainage character begins to effect its color and turbidity. Streams tributary throughout its entire length contribute color. The drainage basin of the Columbia River is approximately 250,000 square miles, divided in six states and British Columbia. There are 13 major streams entering from the north and west and 13 from the south and east. There is insufficient time to enumerate the contributory effect of the streams on the color of the water at any particular point. The following data will present some facts on the color contribution.

At the Dalles, the low discharge of the Columbia may be assumed to be 60,000 second feet and at flood, 700,000 second feet, with a low color measurement of 2 and high of 60. The Spokane River with a low flow of 2,000 and high of 25,000 second feet, yields low color of from 7 to 12. The Okanogan River yields color annually from 8 to

30. The Wenatchee River, with somewhat less annual discharge than the Spokane River, yields color also from 8 to 30. The Naches River, discharging into the Yakima, with a forested drainage basin begins to contribute a larger percentage of color. Color tests vary from 6 low to 60 high. Thus, the effect on the Yakima River is marked; for, at Prosser, the normal color of the Yakima River of 10 above the confluence of the Naches River has increased to a high reading of 200, with an average well above 40. The Snake River, one of the largest tributaries of the Columbia, has a mean discharge of 15,000 to a high water discharge of 220,000 second feet and, in addition, contributes color from low reading in October and November of 8 to a high reading of 140 in March. It may, therefore, be of some importance in certain particular cases, for these contributions to be taken into account in the location of industries affected thereby.

Outside of the Columbia River Basin and with particular attention directed to streams on the Western slope of the Cascades, we find colored water of high degree of concentration. Humus material accumulated in the forested areas furnishes high color concentrations with noticeable daily fluctuations during flood stages or periodic heavy precipitation. Color readings in the following streams are important:

| STREAM | HIGHEST COLOR |
|---|---|
| | *p.p.m.* |
| Cedar River | 60 |
| Chehalis River | 430 |
| Green River | 35 |
| Skagit River | 50 |

The Chehalis River, with a very high visible color as seasonal cycles, traverses heavily forested sections. Conditions of the surface soil, heavily laden with forest products undergoing decomposition, seem to be favorable for high color.

Prediction as to the amount of color to be expected during the seasonal cycles may be made with fair degree of accuracy. The color, more often, is a function of the weather and high water. The periods seem to be in late spring or early summer and later again in the late autumn. Freshets stir up the organic accumulations and they are usually concomitant with heavy rains and melting of snow.

In the area south of the Columbia River channel the rivers of Oregon

may be looked upon as potential sources of color concentration. The Deschutes River flows through lava regions and much of the drainage area is timberless. Notwithstanding this seeming lack of color sources within the drainage area, it is observed that the color reaches as high as 70, when the Deschutes reaches the Columbia. As the River rises in lakes in the Cascade Range, some 5,000 or 6,000 feet above sea level, the source may be considered melted snow Traversing densely forested areas, it finally emerges into the prairie. The fact that the color is practically nil until it reaches the timberless channel is worthy of consideration. Irrigation development in this section with attendant farm grown products appears to contribute to color content.

The Bull Run River is interesting in that its quality is exceptionally high and contains almost no organic color. The theory is that its drainage through an igneous gorge is restrained by glacial moraine, which allows percolation only, filtering naturally. Other geological features restrict the contamination from the Mt. Hood glaciers and the heavy turbidities do not reach the River. Much of the basin is heavily forested, but its character is such that no color is dissolved tributary to the River.

The Willamette River is not so fortunate, even though rivers such as the McKensie supply a very low color contribution. The Willamette River at Salem shows a reading as high as 60 and at Oregon City as high as 150. The McKensie River, like the Bull Run River, seldom shows over 10. The Santiam seldom shows a color reading to exceed 8. Smaller tributaries and extensive logging operations and floating wastes have greatly contributed color to the lower Willamette. These contributory agencies have a marked effect on the paper industry where bleaching operations are required to counteract partially the color absorption.

Smaller streams in the Coastal area on the Oregon Coast are highly colored. Pony Creek at Marshfield shows variations of from 60 to 180; seldom less than 50. Heavy rains in this locality are coincident with high color observations. This supply is interesting in that it averaged the highest color content of any municipal domestic water supply on the Coast. If due thought is given to the conditions in these natural streams, we venture to suggest that developing of water supplies of high quality may be seriously affected by artificial impounding or diversion.

## NATURE OF COLOR

The nature of this color pigment is not easily determined. Qualitative and quantitative readings are limited to range of pigmentation and intensity. These determinations did not contribute any clue as to its chemical or physical properties. Norcom in 1924, conducted important studies on colored waters at Wilmington, N. C. Under the general term of "Coagulation" it was assumed that color and suspended matter would be removed. It was found, however, that color removal was only partially responsive to floc absorption. T. B. Miller, in 1925, made detailed studies of colored solutions. The studies led to experiments on organic soil acids yielding colors comparable to natural conditions. Color from lignin, tannates, gallates and humus are characterized as colloids. Experimentation by dialysis, cataphoresis and chemical reagents sought to determine whether the color was a true solution or a colloid. The importance of these results is obvious, since the type of reaction would be different. Miller found in the cataphoresis experiments the coloring matter was stable and bore a negative charge and behaved as a typical negatively charged colloid. His dialysis results showed that the color failed to pass the membrane, indicating colloid dimensions, and behaved as a negatively charged colloid of the suspensoid type. Further chemical treatment indicated that the addition of a hydroxyl ion only further stabilized the colloid and did not effect clarification. It was found that alum or aluminum sulphate, did produce coagulation. Humic acid was found to respond to this reagent. With small quantities of alkali present, the aluminum sulphate appears to first combine with the alkali, after which, the alum completes the coagulation. We will have occasion to refer to this reaction and its practical application later. The reaction above referred to now gives the visible alum floc, but is positively charged. Furthermore, this reaction takes place at a low pH, about 5.5 or at least between 5 and 6. It was concluded from the results that the coloring matter was precipitated by the aluminum ion and not by the colloidal aluminum hydroxide. When stability is reached, the combination of the two colloids bears a positive charge. These experiments suggested a practical solution of the clarification of highly colored waters and it is significant that the process has been preëminently successful in its practical application.

## DECOLORIZATION METHODS

Various methods of decolorization of waters have been tried with varying success. The attempt has been made to apply the methods of industrial decolorization. The use of activated carbon offers possibilities and is a revival of the early method of purification by charcoal. The limiting feature is that the porous structure absorbing such color and turbidities becomes saturated and only by either strong oxidizing agents or heat can the coloring matter be destroyed and the carbon revivified. This has not reached the stage of practical application for water supply clarification.

Chemical treatment with oxidizing agents, such as chlorine, has been attempted in an effort to oxidize the color colloid to a colorless derivative. The nature of the color colloid is such that there is no encouragement for success using this method. Lignins, associated with wood products, in combination with cellulose base, do appear to bleach practically by this method. The color colloid in natural water does not fall within this classification.

Recently, the use of ferric chloride has shown promise of excellent clarification. The reaction is comparable to that described herein under aluminum ion combination with the color colloid. Ferric chloride requires more delicate control in that the point of optimum clarification takes place at a lower pH and consequently higher acidity. Also, at conditions above the optimum, or higher pH, colloidal iron, as ferric hydroxide, may be carried into the filtrate, a condition which limits its scope of application, especially where iron free water is not only desired, but necessary.

## EXPERIENCE AT MARSHFIELD, OREGON

The problem of clarification at Marshfield, Oregon, is interesting in that the high color was objectionable in appearance in the domestic water supply. In addition, the water was corrosive, augmenting the natural color with red oxide of iron. No less important was the destructive action on the water mains and industrial pipe lines. Pony Creek is a stream with very limited drainage area, depending largely on heavy rain fall with no major tributaries. The drainage area is heavily forested, the surface of which is heavily matted with mould and forest products under decomposition.

Daily readings of color over a period of years showed a very high average, one month averaging 140. This is a strong, tea like, dark brown color.

In 1925, the Water Company requested a report for possible means of clarification. Experiments were conducted, taking as a basis an average water sample from Pony Creek showing color of 140. Following closely the data of Miller's, it was found that, using 1.73 grains of aluminum sulphate and 0.06 grain of lime, as calcium hydroxide, the best and most efficient clarification was obtained at a pH of 5.2. The original sample showed a pH of 6.4. In a period of one hour settling, the color of the filtered water showed color of less than 3. It was also found that the alum could be raised to 2.2 grains per gallon and lime to 0.07 grain per gallon which gave a clarified effluent with a pH of 5.3.

A series of samples was taken of the water supply at various seasons which showed an average pH of 6.3. It was noticeable that when the color reached 170 the pH was 6.1. All the plotted results clearly indicated that the increased color showed a slightly increased acidity. However, the range was not such as to greatly affect the amount of chemical necessary for complete clarification. Based on the results of these experiments, a treating plant was constructed, involving the application of aluminum sulphate and lime at the intake, mixing and sedimentation; filtration and aeration followed. It was interesting to note that the exact dosages found by experimentation produced complete color absorption and the formation of a color floc at the sedimentation basin so that it was visible to any observer that the color had been completely separated from the main water supply and was visible in the darkly stained floc only.

Owing to the fact that this supply carried no turbidities whatever, the resulting floc was not as heavy in gravity as it would have been had there been present a certain amount of occluded suspended matter. Therefore, the filtration plant was designed to take care of any floc which might remain in suspension after 4 hours of sedimentation.

In order to correct the low pH necessary to effect color clarification, additional lime was added after filtration and prior to aeration. The alkalinity was increased to a pH of 7.5 by the addition of 0.25 grain of lime. While the matter of correcting the corrosive qualities of the water is important, it is quite incidental to the main feature of color removal. The point to be emphasized is that freedom from highly colored waters in localities of abundant rainfall cannot be guaranteed in the light of our present data. If corrective measures are necessary for clarification it is also apparent that provision, without additional expense, can be made for decolorization.

It is safe to predict, with our present knowledge of the major water supplies and tributaries in the Coastal States, that the color content will not diminish. On the other hand, our recent investigations have shown marked increases in average readings. It is evident that deforestation, clearing of water sheds, irrigation and drainage will be factors greatly influencing the mineral content, as well as the organic color quality, of our surface water supplies.

# TENTATIVE DRAFT[1] OF STANDARD SPECIFICATIONS FOR STEEL STANDPIPES AND ELEVATED TANKS

SYMBOL: 7Ha—1931

At the annual meeting of the Central States Section in Youngstown, Ohio, on September 25, 1930, Sub-Committee No. 7-H of the Water Works Practice Committee submitted a preliminary report in the form of a tentative draft of "Standard Specifications for Steel Standpipes and Elevated Tanks."

The Sub-Committee consists of Mr. Louis R. Howson, Chairman, Mr. Chas. W. Sherman, and Mr. N. T. Veatch, Jr. Mr. Howson presented this report in person, and it was discussed by five invited engineers and representatives of manufacturers of standpipes and elevated tanks, as well as by various other members present.

Following that meeting, the Sub-Committee revised its draft of the specifications, and again circulated it for comment among the men who had previously discussed it.

The revised tentative draft of the specifications is now here published together with the discussions submitted by the following:

Mr. Alfred O. Doane, Senior Mechanical Engineer, Water Division, Metropolitan District Commission, Boston, Mass.

Mr. George T. Horton, President, Chicago Bridge and Iron Works, Chicago, Ill.

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[1] Published for the purpose of eliciting criticism and suggestions, and subject to revision.

This tentative draft of the specifications as given in the following pages as adopted by Sub-Committee 7-H on Steel Standpipes and Elevated Tanks, L. R. Howson, Chairman, was referred to Mr. G. Gale Dixon, Chairman, Committee No. 7, Transmission and Distribution, and approved by him April 17, 1931. It was then referred by Mr. Dixon to the Committee on Water Works Practice, Malcolm Pirnie, Chairman, and at a meeting of the Committee on Water Works Practice at Pittsburgh, Pa., on May 25, 1931, action was taken directing that the tentative specification be sent out to the members of the Committee on Water Works Practice for letter ballot on its adoption as a tentative standard. The letter ballot was favorable and was so reported on August 4, 1931 to the members of the Committee on Water Works Practice and to the officers of the American Water Works Association.

Mr. J. O. Jackson, Manager, Engineering Department, Pittsburgh Des Moines Steel Company, Pittsburgh, Pa.

Mr. Lyle Merrick, Chief Engineer, Petroleum Iron Works Company, Sharon, Pa.

Mr. H. A. Sweet, Engineer, Associated Factory Mutual Fire Insurance Companies, Boston, Mass.

*Further written discussion on all details concerned with the fabrication and erection of steel standpipes and elevated tanks is desired from all interested members.* The discussions received following this publication will be referred to the Sub-Committee, who will then make a final revision of the tentative draft, and submit a final report recommending adoption of the specifications as the standard of the American Water Works Association for this class of work.

October 7, 1931.

G. GALE DIXON, *Chairman*, Committee 7, Transmission and Distribution.

*Approved:*

MALCOLM PIRNIE, *Chairman*, Committee on Water Works Practice.

## REPORT OF SUBCOMMITTEE 7-H

Mr. G. GALE DIXON, CHAIRMAN
Committee No. 7, Transmission and Distribution
American Water Works Association
Youngstown, Ohio

*Dear Sir:*

The undersigned Sub-Committee 7-H, has for the past year been engaged in drafting a "Standard Specification for Steel Standpipes and Elevated Tanks." In the preparation of this specification, the Committee has been materially assisted by the work of the prior committee under the chairmanship of Mr. James E. Gibson, who kindly placed at the disposal of this Committee the results of work done by his Committee, and by the constructive criticisms and suggestions of various members of the water works profession, including particularly Mr. George T. Horton of the Chicago Bridge and Iron Works, Messrs J. E. O'Leary and J. O. Jackson of the Pittsburgh-Des Moines Steel Company, Mr. Lyle Merrick of the Petroleum Iron Works, Mr. H. A. Sweet, Engineer of the Associated Factory Mutual Fire Insurance companies, Mr. Alfred O. Doane, Senior Mechanical

Engineer, Water Division, Metropolitan District Commission of Boston, and several consulting engineers who have furnished us copies of specifications issued by their respective organizations.

The following specifications are submitted by the Committee as its conclusion after much study and discussion of opinions and practice in the design and construction of Steel Standpipes and Elevated Tanks.

Respectfully submitted,
CHARLES W. SHERMAN,
N. T. VEATCH, JR.,
L. R. HOWSON,
*Chairman Sub-Committee 7-H.*

## STANDARD SPECIFICATIONS FOR STEEL STANDPIPES AND ELEVATED TANKS

Symbol: 7Ha—1931

The American Water Works Association recommends these specifications as a uniform guide in the preparation of the designs, the definition of materials, stresses, workmanship, fabrication and methods to be followed in the erection and the construction of standpipes, elevated tanks and supporting towers. It is recognized in presenting these specifications that construction of this type must necessarily be adapted to a wide variation in conditions and that no specification can be written to cover all the details of those variations. It is hoped that the design, materials and workmanship will in all cases be at least equal to the requirements herein set forth.

### *1. Definitions*

Where the word "standpipe" is used in these specifications, it is understood to mean a tank having a cylindrical shell and a flat bottom resting upon a prepared foundation located directly upon the ground. It may or may not be anchored to this foundation, depending upon its proportions.

Where the word "elevated tank" or "tank" is used, it is understood to mean a cylindrical shelled tank supported upon an elevated structure or tower, to which it is securely anchored, so that the tank and its supporting structure act as a unit.

Where the word "tower" is used, it is understood to mean the supporting steel frame work or other structure supporting an elevated tank.

By "tower height," it is understood to mean the vertical distance measured from the top of the foundation pier or piers to the bottom capacity line of the elevated tank.

*2. Information to be furnished by purchaser*

The purchaser in asking for proposals, should furnish the bidders with complete information as to local conditions, location and size of pipe connections, nature of soil upon which foundation is to rest, location of the site of erection with respect to railroad sidings and points at which materials can be unloaded. Purchaser shall also advise definitely the construction and materials which will be furnished to the contractor, the capacity of standpipe or tank required and minimum and maximum dimensions.

*3. Capacity*

The standard sizes for standpipes and elevated tanks shall be those recommended by the Division of Simplified Practice of the Department of Commerce and shall be as follows:

| | |
|---|---|
| 5,000 | 100,000 |
| 10,000 | 150,000 |
| 15,000 | 200,000 |
| 20,000 | 250,000 |
| 25,000 | 300,000 |
| 30,000 | 400,000 |
| 40,000 | 500,000 |
| 50,000 | 750,000 |
| 60,000 | 1,000,000 |
| 75,000 | 1,500,000 |
| | 2,000,000 |

All in United States gallons net capacity

## ELEVATED TANKS

### *Design and materials*

*4. Form and proportions*

Elevated tanks shall be cylindrical in cross-section and may have a flat bottom supported on a grillage of "I" beams and concrete, but ordinarily shall have a suspended bottom of conical, spherical or ellipsoidal shape, or a supported bottom made up of a composite of the above surfaces.

The total height of an elevated tank measured from the lowest

point of the bottom of the overflow level should not be more than 1½ times the diameter of the tank, except in unusual cases.

*5. Anchorage*

The tower columns shall be attached to the foundations by means of anchorage bolts capable of resisting the greatest uplift that can be produced by the wind load with stress in bolts not exceeding 15,000 pounds per square inch.

*6. Towers*

The height of towers shall be that necessary to meet the requirements of elevation as requested by the purchaser in asking for bids.

No elevated tank shall have less than four columns in its supporting tower. Four post towers may be used for elevated tanks up to 150,000 gallons capacity, providing the diameter of the tank is such that the distance between columns at the top does not exceed 20 feet. For larger tanks, six or more columns shall be used.

In order to avoid unnecessary eccentric loading on the columns and local stresses in the tank bottom, the connections between the columns and the side of the tank shall be made in such a manner that the center of gravity of the column section intersects as closely as possible, the center of connection between the bottom and the sides of the tank. The columns shall be designed for any eccentricity. Enough rivets shall be provided above this intersection to transmit the total column load.

Tops of inclined columns shall be connected by a continuous, horizontal circular girder if the tank bottom is suspended. The outer flange should preferably be symmetrical about the web plate, but if eccentric, the allowable working stresses for the flanges shall be reduced 25 percent. The bending moment shall be considered as carried entirely by the flanges and web in proportion to their areas at the section considered. Splices shall be designed to transmit the load in the spliced member. The horizontal component of the maximum column load shall be transmitted to the girder without subjecting the rivets to tension. The circular girder may be used as a balcony. Drain holes shall be provided in the web plate.

If the columns are vertical and the tank bottom suspended, the circular girder shall be as stated in the preceding paragraph, or else shall be a structural shape connected to the tank by the rivets of the joint between the shell and bottom, the shape extending as close as possible to the columns.

Tower columns may be vertical or inclined as conditions may warrant, but for aesthetic design, inclined legs are generally to be preferred, the minimum batter or inclination being one twenty-fourth of the height. Greater inclination may be desirable for high towers in order to obtain greater stability.

*7. Diagonal bracing in the tower*

The vertical supply pipe shall be thoroughly braced laterally with adjustable diagonal bracing at the panel points of the tower.

The diagonal bracing in the tower shall be calculated for an initial stress of 3,000 pounds per square inch, in addition to wind stresses, etc. Diagonal sway bracing shall be by riveted or adjustable rod members. Adjustable rod members may be used in tanks of less than 500,000 gallons capacity. On larger tanks, sway bracing shall be by riveted members.

*8. Balcony*

In the case of batter post towers, provision shall be made at the junction of the tower with the tank to take care of the horizontal thrust due to the inclination of the columns. With towers over 50 feet in height, this shall be cared for by a balcony which shall be constructed about the tank at the level where the cylindrical portion joins the bottom. The balcony shall be at least 24 inches wide for tanks up to 15 feet in diameter and not less than 30 inches for tanks of larger diameters. Balcony shall have a floor plate with a minimum thickness of $\frac{5}{16}$-inch suitably punched for drainage. The balcony shall be provided with a structural steel hand railing at least 3 feet high constructed so as to stiffen the outer chord of the girder.

*9. Riser pipe*

The supply or riser pipe shall be riveted or welded steel pipe having an internal diameter depending upon tank size. (Ordinarily the diameter will be from 3 to 4 feet for tanks up to 150,000 gallons and from 5 to 10 feet for larger tanks.) The circumferential joints (which should preferably be riveted) shall be designed to withstand the compressive load imposed by the tank and the vertical joints shall be designed to safely withstand the bursting stress in the riser. If riveted, the joints shall in general conform to the requirements for joints in the steel plate work of the tank. The supply pipe shall extend from the level of the top of the concrete foundations to the bot-

tom of the tank. The lower end of the supply pipe shall be closed by means of a suitable flanged bottom plate $\frac{3}{8}$-inch thick.

The contractor shall furnish and rivet to the bottom plate of the elevated tank riser pipe, a cast steel flange piece of such dimensions and construction that a —— inch, class —— pipe may pass through the casting and a lead joint be poured.

The flanged piece shall have grooves for calking the jute gaskets, shall be riveted or welded to the bottom plate and shall be of design approved by the Engineer. The contractor shall furnish a silt stop (preferably removable suitable for setting into the flange piece previously mentioned.) The silt stop shall be of cast iron or steel, set 6 inches high above the bottom of the riser pipe bottom when in place, and shall be not less than $\frac{3}{8}$ inch thick.

Just above the bottom of the supply pipe and in the side of the riser, there shall be an elliptical manhole having a nominal size of not less than 12 by 16 inches, fitted with a proper cover, gasket and fastening to make it water-tight. The opening shall be suitably reinforced and a suitable crane provided to hold the cover. (On large riser pipes an 18- by 24-inch manhole is desirable.)

The vault in the concrete foundation which supports the bottom of the steel supply pipe, will be built by the Owner, but the contractor shall grout between the steel plate and the concrete after the tank is erected.

*Note.* In elevated tanks designed with riveted or welded steel riser pipe without expansion joint, the riser pipe shall be designed to carry a portion of the load of the suspended bottom to the foundation. The minimum load for the riser pipe foundation shall be considered a column of water the diameter of the riser pipe at its junction with the suspended bottom of the tank and the height of the riser at this point above the foundation, plus a cylinder of water 4 feet greater in diameter than the riser and extending from the top of the riser to the maximum elevation of water in the tank.

*10. Top stiffeners*

On all elevated tanks not provided with a roof or adequately stiffened by the roof or cover, the top of the tank shall be stiffened with structural shapes having a section modulus which in inches shall be at least equal to one two hundred fiftieth part of the square of the diameter in feet. Joints in stiffener members shall be spliced. The thickness of any structural shape making up the stiffener shall be not less than $\frac{3}{8}$ inch.

*11. Overflow and freeboard*

An overflow —— inches in diameter shall be provided. Discharge shall be visible and pitched to drain. Exposed length shall not exceed 4 feet. Sufficient space shall be provided between the roof and the top edge of the tank to permit of the overflow of water in case the tank is overfilled.

*12. Roof*

All elevated tanks used for the supply of water for domestic and sanitary purposes, should be covered. With elevated tanks enclosed by masonry having a roof, no independent tank roof will be required if construction of the enclosure is such as to prevent access of birds or animals to the tank.

The roof shall in all cases provide for ventilation of the space above the water in the tank, but shall be so designed and constructed as to prevent the ingress and egress of birds, bats or similar animals.

The roof shall be either self-supporting or if the diameter and pitch require, shall be supported by suitable angle or channel rafters or by other satisfactory means. The roof plates shall have a thickness of not less than $\frac{3}{16}$ inch.

There shall be an ornamental finial on the top of the roof sufficiently strong to support a revolving ladder and a painter's hoist.

A trap door about 2 feet square shall be provided in the roof plate conveniently arranged to be operated from the ladder.

*13. Ladder*

A steel ladder shall be provided for the elevated tank, supported by one of the tower columns and shall extend from a point 12 feet above the top of the concrete foundations to the top of the balcony rail. The side members of the ladder shall be bent over and fastened to the balcony or tank in such manner as to form suitable hand rails. On tanks provided with a roof there shall be a ladder extending from the level of the balcony to the peak of the roof so constructed and fastened to the finial in the peak of the roof and running on rollers as to permit the ladder being swung around the tank. On tanks without a roof there shall be a fixed ladder extending from the balcony to three feet above the top of the tank shell the side members to be constructed in such manner as to form suitable hand rails. The side members of the ladder shall be not less than $\frac{3}{8}$ by 2 inches, spaced 15 inches apart, the rungs not less than $\frac{5}{8}$ inch square, spaced 12 inches

apart, set with the edge on top and the brackets not less than ½ by 3 inches, spaced not to exceed 12 feet apart.

*14. Materials*

Steel for plates and rolled shapes shall conform to the "Standard Specifications for Structural Steel for Bridges," of the American Society for Testing Materials, Serial Designation A-7 Structural Steel, of the latest revision.

Steel for deformed tank bottoms shall be of pressing quality, the physical properties being the same as for rivet steel, as cited in Specifications A-7 for Structural Steel, latest revision.

Rods shall be of mild steel suitable for upsetting, welding and threading.

Copper bearing steel or other material resistant to air corrosion shall be used for plates in balcony and roof.

Copper bearing steel shall contain not less than 0.2 percent copper. In all other respects, it shall conform to the above standard specifications.

*15. Requirements for design*

The loads to be used for designing the tank, tower, and foundations shall be as follows:

*Dead load.* The dead load shall be the estimated weight of all permanent construction and fittings.

*Live load.* The live load shall be the weight of the liquid contents of the tank when full. If the roof has a slope of less than 25 degrees with the horizontal, the live load shall also include a snow load of 25 pounds per square foot on the horizontal projection of the roof for all tanks built in locations where snowfall occurs. The design of the tower need not include the weight of the water contained in the large steel riser which shall be considered as a cylinder extending from the water surface to the base of the riser, the diameter of the cylinder being taken as that of the riser at the foundation.

*Wind load.* Wind pressure shall be assumed to be 30 pounds per square foot on a vertical plane surface or a maximum of 50 pounds per lineal foot of column. In calculating the wind load on the cylindrical surface six-tenths of the above pressure shall be applied to the total area of the vertical projection, and the point of application of the load shall be at the center of gravity of the projected area. The load on the tower shall be assumed to be concentrated at the

panel point. Wind load on the vertical projected area of spheroidal or conical surfaces shall be 15 pounds per square foot. Stresses due to wind may be neglected if they are less than 25 per cent of the dead and live load.

In locations exposed to unusual winds, the assumed wind pressure shall be taken at 40 pounds per square foot.

*Balcony and walkway loads.* Balcony shall be designed to carry a concentrated load of 1000 pounds at any point without overstressing.

*16. Allowable stresses*

All parts of the structure shall be designed to resist safely the maximum stresses that can be provided by any combination of the above loads. The maximum stresses in pounds per square inch produced by the above loads shall not exceed the following:

| | *pounds per square inch—net section* |
|---|---|
| *Tension:* | |
| Tension in cylindrical tank plates | 15,000 |
| Tension in bottom course and formed bottom plates | 12,000 |
| *Shearing:* | |
| Rivets in cylindrical portion of tank | 11,250 |
| Rivets in bottom course and formed bottom plates | 9,000 |
| Rivets in main column connection to tank | 7,500 |
| *Bearing:* | |
| Bearing on rivets shall be twice the allowable shear. | |
| *Compression:* | |
| Compression in steel | 18,000 |

For compression members, the permissible unit stress of 18,000 pounds per square inch shall be reduced by the formula:

$$p = \frac{18{,}000}{1 + \frac{L^2}{18{,}000\, r^2}}$$

where $p$ equals permissible working stress in compression in pounds per square inch; $L$ = length of member from center to center of connections in inches; $r$ = least radius of gyration of section in inches.

The ratio $\frac{L}{r}$ shall not exceed 125 for main members and 175 for struts and roof connection members.

*17. Basis of stress computations*

The pressure possible at the bottom of each course in the cylindrical

portion of the tank shall be assumed as constant throughout such course, and the tensile stress computed accordingly.

The tensile stress in tank bottoms shall be assumed as constant throughout each course and equal to the maximum stress intensity that may exist anywhere in such course.

*18. Net sections*

Net sections shall be used in calculating the tensile strength in plates and members. In deducting rivet holes, their diameter shall be taken $\frac{1}{8}$ inch larger than the undriven rivets for punched holes and $\frac{1}{16}$ inch larger for reamed or drilled holes.

*19. Minimum thickness*

The minimum thickness of metal in the tank shall be $\frac{1}{4}$ inch for tanks up to and including 500,000 gallons capacity. For larger sizes $\frac{3}{8}$ inch shall be minimum thickness.

The minimum thickness of metal in towers shall be $\frac{1}{4}$ inch except in the webs of channels and "I" beams and in minor parts which carry no load.

The minimum thickness of cover plates shall be $\frac{1}{4}$ inch, but not less than one-fortieth of the distance between rivet lines. A cover plate may be used on but one side of a column.

*20. Rivets*

The diameter of rivets shall be not less than the thickness of the thickest plate connected in any joint. The minimum permissible diameter of rivets shall be $\frac{5}{8}$ inch in tank plates.

The rivets used shall be of the proper length and diameter to completely fill the rivet holes and form a strong and satisfactory hemispherical or rounded conical head concentric with the rivet holes. Rivets shall have diameters within $\frac{1}{16}$ inch of the diameter of the rivet holes in which they are to be placed.

*21. Spacing of rivets*

The minimum rivet pitch shall be not less than three times the nominal diameter of the rivet.

The maximum pitch along a calked edge, except for column connections, shall not exceed 2.5 times the thickness of the thinnest plate connected for single rivet joints or 3.0 times the thickness of the

thinnest plate in joints having more than one row of rivets, plus, in each case, the diameter of the rivet hole in inches plus 1.5 inches.

Maximum pitch along uncalked edges shall not exceed thirty times the thickness of the thinnest plate connected.

*22. Edge distance*

The distance between the center of the outer row of rivets and the edge of plate shall not be less than one and one-half times the diameter of the rivets. Where edges are calked, the distance shall not exceed one and three-fourths times the diameter of the rivets.

*Workmanship*

*23. Shearing*

All shearing shall be neatly and accurately done. All cuts shall be clean without drawn or ragged edges and without splitting away from the sheared edge. Calked edges may be bevel-sheared on plates $\frac{1}{2}$ inch thick and under. Plates over $\frac{5}{8}$ inch thick shall have calking edges planed.

*24. Beveling*

The edges of all plates and butt straps which are to be calked, shall be beveled to an angle of approximately 70 degrees with the plane of the plate for plates up to $\frac{1}{2}$ inch in thickness and for heavier plates, the angle shall be increased to approximately 80 degrees.

*25. Punching, reaming and drilling*

Rivet holes in material $\frac{1}{2}$ inch thick and under may be punched or drilled full size.

Rivet holes in materials over $\frac{1}{2}$ inch to and including $\frac{3}{4}$ inch thick and in thinner plates for butt joints shall be either drilled from the solid or punched $\frac{1}{8}$ inch smaller in diameter than the nominal diameter of the rivet and then reamed to size.

Rivet holes in material over $\frac{3}{4}$ inch thick shall be drilled.

For the butt joints, rivet holes shall be laid out the required distances from the machined edges of the plates, and where the edges are beveled, the distances shall be laid out from the edge of the surface of the plate with which the rivet heads will be in contact.

The edges of the rivet holes which will be in contact with the rivet heads shall be beveled $\frac{1}{16}$ inch with a twist drill or a rose reamer.

Punched rivet holes shall be made with a punch whose diameter

shall not exceed the diameter of the rivet more than $\frac{1}{16}$ inch. The diameter of reamed and drilled holes shall not be more than $\frac{1}{16}$ inch larger than the rivet.

All punched and reamed holes shall be clean cut without torn or ragged edges. Rivet holes shall be accurately spaced. Any noticeable burrs shall be removed by a tool countersinking not more than $\frac{1}{16}$ inch.

*26. Shaping plates and angles*

The plates, straps, angles and other structural shapes shall be accurately shaped to the forms required. All plates, butt straps and structural shapes shall be accurately cold rolled to the radius of the course for which they are intended. The curvature of any plate, angle or other structural shape shall be uniform throughout without flat places at the ends.

Plates to be scarfed may be heated to a cherry red color, but not hot enough to ignite a piece of dry hard wood when applied to it. Most careful attention shall be paid to all scarfing. No scarfing shall be done at a black heat.

*27. Scarfing*

Where three thicknesses of metal overlap the corner or end of interior plate shall be scarfed to a thin edge. The beveled surface shall be smooth and shall exceed in length eight times the plate thickness.

*28. Protection of steel*

All structural steel received on the work shall be immediately placed upon substantial shores or blocking of sufficient size and strength to prevent any metal from touching the ground, and in such a manner that water cannot collect thereon so that the material will be protected against bending under its own or superimposed loads. Any steel members which may become bent shall be replaced by new material. Before assembling, the surfaces to be in contact with each other shall be thoroughly cleaned.

*29. Setting columns*

The columns shall be set to exact elevation and position on wedges and while thus supported, the spaces beneath shall be thoroughly filled with Portland cement grout. The nuts on all anchor bolts

shall be screwed down tight and the tops of the bolts up-set sufficiently to prevent the nuts from becoming loose.

*30. Assembling and erecting*

All burrs, projections, dirt and rust shall be removed from faying surfaces before the plates are assembled, and the plates shall be securely bolted at the joints in advance of the riveting so as to be in close contact before the rivets are driven.

All the rivet holes in the plates shall coincide, and any holes which do not do so shall be reamed, and larger rivets shall be used where necessary. The use of drift pins will be allowed only for bringing together the several parts forming part of the structure, and force shall not be used to fair the rivet holes, under any circumstances.

If, in the opinion of the Engineer, the holes in any plate cannot be made to coincide without weakening the joint or producing unsatisfactory work, the contractor shall provide a new and satisfactory plate.

The use of sledges in shaping any part of the structure shall not be allowed. The use of sledges in erecting tank structures shall be limited to the necessary laying up or forging to close the shell plates around scarfed corners or butt straps. Care shall be taken to prevent material from falling, or from being in any way subjected to heavy shocks.

*31. Riveting*

The field formed head of rivets shall be on the same side of the tank as that on which the calking will be done. Heads shall present a uniform appearance.

The rivets shall be driven so as to pinch the plates firmly, and all loose or defective rivets or rivets with badly shaped heads or with heads out of position, shall be cut out and replaced in a satisfactory manner.

The rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

*32. Calking*

Tank shall be made tight by calking. No foreign materials shall be placed between the surface of overlaps.

Calking shall be done with a round nose tool and only by experienced and skilled men.

*33. Testing*

After the structure has been erected (but prior to painting), and all seams have been calked, it shall be filled with water furnished by the owner and shall be tested for water-tightness. All leaky seams shall be calked by the contractor, to make the tank watertight. Leaky rivets shall be cut out and replaced by new ones. Any defects revealed by this test shall be made good by the contractor in a satisfactory manner. Tests for water-tightness shall be repeated until the tank is perfectly tight. The contractor shall guarantee the tightness of the elevated tank.

*34. Painting*

All steel shall be cleaned and painted in accordance with the Specifications for Painting Standpipes adopted by the New England Water Works Association at its 1930 annual convention held in Atlantic City.

The specifications are printed in full herein following the specification for Standpipes by permission of the New England Water Works Association.

[Note: The purchaser shall state which of the methods of cleaning covered by this specification he desires and which of the paints he wishes applied. (See Note 1 on page 2163 of the above mentioned specification.) It is believed that the most effective method of cleaning is by sandblast and that the most satisfactory paint for the interior of the standpipe or elevated tank is the ready mixed red lead paint, as therein specified.]

## STANDPIPES

### *Design and materials*

*35. Forms and proportions*

Standpipes shall be cylindrical in cross-section and shall have a flat bottom resting directly on foundation.

Attention is called to the fact that a standpipe or flat bottom tank having a vertical height much greater than the diameter of the tank shell, is not an economical form or proportion. For service purposes and maintenance of uniform pressure on the distribution system, it is always desirable to make the diameter as large as possible, thereby obtaining the desired capacity with a minimum change in elevation of water surface.

*36. Anchorage*

The standpipe shall be attached to the foundation by means of anchorage bolts in all cases where any net uplift or overturning moment exists when conditions are such that the standpipe is empty and full wind pressure is exerted. If uplift exists, anchors shall be not less than $1\frac{1}{4}$ inches in diameter set deep enough to take the uplift and of sufficient cross-section that the unit stress will not exceed 15,000 pounds per square inch under maximum conditions. Anchor bolts shall be in even multiples of the number of plates forming the lower ring of the standpipe. Anchor bolts shall be attached to the cylinder by properly designed lugs made of structural steel or bent plates. Cast iron lugs shall not be used.

*37. Pipe connections*

The contractor shall furnish and rivet to the bottom plate of the standpipe a plate $\frac{1}{2}$ inch thick around the opening for the inlet pipe connection and a cast steel flange piece of such dimensions and construction that a —— inch, class —— pipe may pass through the casting and a lead joint be poured.

The flange piece shall have grooves for calking the jute gaskets, shall be riveted or welded to the bottom plates and shall be of design approved by the Engineer. The contractor shall furnish removable silt stop suitable for setting into the flange piece previously specified. The silt stop shall be of cast iron or steel, shall set 6 inches high above the bottom of the standpipe when in place, and shall be not less than $\frac{3}{8}$ inch thick.

*38. Manhole*

Just above the bottom in the side of the standpipe, there shall be an elliptical manhole having nominal size of not less than 18 by 24 inches, fitted with a proper cover, gasket and fastening to make it watertight. The opening shall be suitably reinforced and a suitable crane provided to hold the cover.

The vault, built in the foundation which supports the bottom of the standpipe, will be built by the owner, but the contractor shall grout between the steel plate and the concrete after the standpipe is erected.

*39. Top stiffeners*

On all standpipes not provided with a roof or adequately stiffened

by the roof or cover, the top of the standpipe shall be stiffened with structural shapes having a section modulus which in inches shall be at least equal to one two hundred fiftieth part of the square of the diameter in feet. Joints in stiffener members shall be spliced. The thickness of any structural shape making up the stiffener shall be not less than $\frac{3}{8}$ inch.

*40. Overflow and freeboard*

An overflow —— inches in diameter shall be provided. Discharge shall be visible and pitched to drain. Exposed length shall not exceed 4 feet. Sufficient space shall be provided between the roof and the top edge of the standpipe to permit of the overflow of water in case the standpipe is overfilled.

*41. Roof*

All standpipes used for the supply of water for domestic and sanitary purposes, should be covered. With standpipes enclosed by masonry having a roof, no independent standpipe roof will be required if construction of the enclosure is such as to prevent access of birds or animals to the standpipe.

The roof shall in all cases provide for ventilation of the space above the water in the standpipe, but shall be so designed and constructed as to prevent the ingress and egress of birds, bats or similar animals.

The roof shall be either self-supporting or if the diameter and pitch require, shall be supported by suitable angle and channel rafters or by other satisfactory means and shall be raised above the top of the cylindrical portion of the tank a sufficient amount to allow a painter's trolley to operate. The roof plates shall have a thickness of not less than $\frac{3}{16}$ inch.

There shall be an ornamental finial on the top of the roof sufficiently strong to support a revolving ladder and a painter's hoist.

A trap door about 2 feet square shall be provided in the roof plate conveniently arranged to be operated from the ladder.

*42. Ladder*

On standpipes built without a roof a steel ladder shall be provided extending from a point 12 feet above the top of the concrete foundation to 3 feet above the top of the shell. The side members of the ladder shall be bent over and fastened to the standpipe shell in such manner as to form suitable hand rails. On standpipes having a

roof there shall be provided a ladder extending from a point 12 feet above the top of the concrete foundation to the peak of the roof so constructed and fastened to the finial in the peak of the roof and running on rollers as to permit the ladder being swung around the standpipe. The side members of the ladder shall be not less than $\frac{3}{8}$ by 2 inches, spaced 15 inches apart, the rungs not less than $\frac{5}{8}$ inch square spaced 12 inches apart, set with the edge on top and the brackets not less than $\frac{1}{2}$ by 3 inches, spaced not to exceed 12 feet apart.

*43. Materials*

Steel for plates, rivets and rolled shapes shall conform to the "Standard Specifications for Structural Steel for Bridges," of the American Society for Testing Materials, Serial Designation A-7, Structural Steel, of the latest revision.

All steel to be used for plates for the roof shall be of copper bearing steel or other material resistant to air corrosion. Copper bearing steel shall contain not less than 0.2 percent copper. In other respects, it shall conform to the above Standard Specifications.

*44. Requirements for design*

The loads to be used for designing the standpipe and foundations shall be as follows:

*Dead load.* The dead load shall be the estimated weight of all permanent construction and fittings.

*Live load.* The live load shall be the weight of the liquid contents of the standpipe when full. If the roof has a slope of less than 25 degrees with the horizontal, the live load shall also include a snow load of 25 pounds per square foot on the horizontal projection of the roof for all tanks built in locations where snow fall occurs.

*Wind load.* Wind pressure shall be assumed to be 30 pounds per square foot on a vertical plane surface. In calculating the wind load on the cylindrical surface, six-tenths of the above pressure shall be applied to the total area of the vertical projection, and the point of application of the load shall be at the center of gravity of the projected area. Stresses due to wind may be neglected if they are less than 25 per cent of the dead and live load.

In locations exposed to unusual winds, the assumed wind pressure should be taken at 40 pounds per square foot.

*45. Allowable stresses*

All parts of the structure shall be designed to resist safely the maximum stresses that can be provided by any combination of the above loads. The maximum stresses in pounds per square inch produced by the above loads shall not exceed the following:

| | *pounds per square inch–net section* |
|---|---|
| *Tension:* | |
| Tension in cylindrical standpipe plates | 15,000 |
| *Shearing:* | |
| Shear on rivets | 11,250 |
| *Bearing:* | |
| Bearing on rivets | 22,500 |
| *Compression:* | |
| Compression in steel | 18,000 |

For compression members, the permissible unit stress of 18,000 pounds per square inch shall be reduced by the formula:

$$p = \frac{18{,}000}{1 + \frac{L^2}{18{,}000\ r^2}}$$

where $p$ equals permissible working stress in compression in pounds per square inch; $L$ = length of member from center to center of connections in inches; $r$ = least radius of gyration of section in inches.

The ratio $\frac{L}{r}$ shall not exceed 175 for roof connection members.

*46. Basis of stress computations*

The pressure possible at the bottom of each course in the standpipe shall be assumed as constant throughout such course, and the tensile stress computed accordingly.

*47. Net sections*

Net sections shall be used in calculating the tensile strength in plates and members. In deducting rivet holes, their diameter shall be taken $\frac{1}{8}$ inch larger than the undriven rivets for punched holes and $\frac{1}{16}$ inch larger for reamed or drilled holes.

*48. Minimum thickness*

The minimum thickness of metal in the standpipe shell shall be $\frac{1}{4}$ inch for standpipes up to and including 500,000 gallons capacity. For larger sizes $\frac{3}{8}$ inch shall be minimum thickness.

The minimum thickness of metal in the bottom plate shall be $\frac{1}{4}$ inch and the maximum $\frac{1}{2}$ inch. Thickness within these limits shall be determined from the formula:

$$T = \frac{H + 125}{600}$$

in which $T$ = thickness in inches and $H$ = height of standpipe in feet. Thickness used shall be taken as nearest $\frac{1}{16}$ inch above result from the formula.

*49. Rivets*

The diameter of rivets shall be not less than the thickness of thickest plate connected in any joint. The minimum permissible diameter of rivets shall be $\frac{5}{8}$ inch in standpipe plates.

The rivets used shall be of the proper length and diameter to completely fill the rivet holes and form a strong and satisfactory hemispherical or rounded conical head concentric with the rivet holes. Rivets shall have diameters within $\frac{1}{16}$ inch of the diameter of the rivet holes in which they are to be placed.

Attention is called to the requirements that copper bearing steel rivets shall be used in the roof and balcony.

*50. Spacing rivets*

The minimum rivet pitch shall not be less than three times the nominal diameter of the rivet.

The maximum pitch along a calked edge shall not exceed $2\frac{1}{2}$ times the thickness of the thinnest plate connected for single rivet joints or 3.0 times the thickness of the thinnest plate in joints having more than one row of rivets, plus, in each case, the diameter of the rivet plus $1\frac{1}{2}$ inches.

Maximum pitch along uncalked edges shall not exceed thirty times the thickness of the thinnest plate connected.

*51. Edge distance*

The distance between the center of the outer row of rivets and the edge of plate shall not be less than one and one-half times the diameter of the rivets. Where edges are calked, the distance shall not exceed one and three-fourths times the diameter of the rivets.

*Workmanship*

*52. Shearing*

All shearing shall be neatly and accurately done. All cuts shall be clean without drawn or ragged edges and without splitting away from the sheared edge. Calked edges may be bevel sheared on plates $\frac{5}{8}$ inch thick and under. Plates over $\frac{5}{8}$ inch thick shall have calking edges planed.

*53. Beveling*

The edges of all plates and butt straps which are to be calked, shall be beveled to an angle of approximately 70 degrees with the plane of the plate for plates up to $\frac{1}{2}$ inch in thickness and for heavier plates the angle shall be increased to approximately 80 degrees.

*54. Punching, reaming and drilling*

Rivet holes in material $\frac{1}{2}$ inch thick and under may be punched or drilled full size.

Rivet holes in materials over $\frac{1}{2}$ inch to and including $\frac{3}{4}$ inch thick and in thinner plates for butt joints shall be either drilled from the solid or punched $\frac{1}{8}$ inch smaller in diameter than the nominal diameter of the rivet and then reamed to size.

Rivet holes in material over $\frac{3}{4}$ inch thick shall be drilled.

For the butt joints, rivet holes shall be laid out the required distances from the machined edges of the plates, and where the edges are beveled, the distances shall be laid out from the edge of the surface of the plate with which the rivet heads will be in contact.

The edges of the rivet holes which will be in contact with the rivet heads shall be beveled $\frac{1}{16}$ inch with a twist drill or a rose reamer.

Punched rivet holes shall be made with a punch whose diameter shall not exceed the diameter of the rivet more than $\frac{1}{16}$ inch. The diameter of reamed and drilled holes shall not be more than $\frac{1}{16}$ inch larger than the rivet.

All punched and reamed holes shall be clean cut without torn or ragged edges. Rivet holes shall be accurately spaced. Any noticeable burrs shall be removed by a tool countersinking not more than $\frac{1}{16}$ inch.

*55. Shaping plates and angles*

The plates, straps, angles and other structural shapes shall be accurately shaped to the forms required. All plates, butt straps and

structural shapes shall be accurately cold rolled to the radius of the course for which they are intended. The curvature of any plate, angle or other structural shape shall be uniform throughout, without flat places at the ends.

Plates to be scarfed may be heated to a cherry red color, but not hot enough to ignite a piece of dry hard wood when applied to it. Most careful attention shall be paid to all scarfing.

*56. Scarfing*

Where three thicknesses of plate overlap the corner or end of interior plate shall be scarfed to a thin edge. The beveled surface shall be smooth and shall exceed in length eight times the plate thickness.

*57. Protection of steel*

All structural steel received on the work shall be immediately placed upon substantial shores or blocking of sufficient size and strength to prevent any metal from touching the ground, and in such a manner that water cannot collect thereon and so that the material will be protected against bending under its own or superimposed loads. Any steel members which may become bent shall be replaced by new material. Before assembling, the surfaces to be in contact with each other shall be thoroughly cleaned.

*58. Assembling and erecting*

All burrs, projections, dirt and rust shall be removed from faying surfaces before the plates are assembled, and the plates shall be securely bolted at the joints in advance of the riveting so as to be in close contact before the rivets are driven.

All the rivet holes in the plates shall coincide, and any holes which do not do so shall be reamed, and larger rivets shall be used where necessary. The use of drift pins will be allowed only for bringing together the several parts forming part of the structure, and force shall not be used to fair the rivet holes under any circumstances.

If, in the opinion of the Engineer, the holes in any plate cannot be made to coincide without weakening the joint or producing unsatisfactory work the contractor shall provide a new and satisfactory plate.

The use of sledges in shaping any part of the structure shall not be allowed. The use of sledges in erection shall be limited to the

necessary laying up or forging to close the shell plates around scarfed corners or butt straps. Care shall be taken to prevent material from falling, or from being in any way subjected to heavy shocks.

*59. Placing bottom of standpipe on foundation*

The bottom shall be supported and lowered upon the foundation furnished by the purchaser in such a manner as to prevent sag.

The standpipe shall be supported upon a cushion of sand and cement approximately 2 inches in thickness, the materials for which will be furnished by the purchaser at the site of the work. Contractor shall thoroughly mix the sand and cement in equal parts dry and shall spread and carefully level it in such a manner as to provide a uniform bearing over the entire bottom of the standpipe. The bottom shall be absolutely level.

After the standpipe is completed, the sand and cement shall be raked out for a depth of at least 6 inches all around the circumference of the standpipe and around the hole in the foundation for the inlet pipe and be replaced by grout or mortar composed of equal parts of Portland cement and clean, sharp sand, thoroughly filling the raked space and finishing neatly with a bevel on the outside.

*60. Riveting*

The field formed head of rivets shall be on the same side of the standpipe as that on which the calking will be done. Heads shall present a uniform appearance.

The rivets shall be driven so as to pinch the plates firmly and all loose or defective rivets with badly shaped heads or with heads out of position, shall be cut out and replaced in a satisfactory manner.

The rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

*61. Calking*

Standpipe shall be made tight by calking. No foreign materials shall be placed between the surface of overlaps.

Calking shall be done with a round nose tool and only by experienced and skilled men.

*62. Testing*

After the structure has been erected (but prior to painting), and all seams have been calked, it shall be filled with water furnished by

the owner and shall be tested for watertightness. All leaky seams shall be calked by the contractor, to make the tank watertight. Leaky rivets shall be cut out and replaced by new ones. Any defects revealed by this test shall be made good by the contractor in a satisfactory manner. Tests for watertightness shall be repeated until the standpipe is perfectly tight. The contractor shall guarantee the tightness of the standpipe.

*63. Painting*

All steel shall be cleaned and painted in accordance with the Specifications for Painting Standpipes adopted by the New England Water Works Association at its 1930 annual convention held in Atlantic City.

The specifications are printed in full herein following the specification for Standpipes by permission of the New England Water Works Association.

[Note: The purchaser shall state which of the methods of cleaning covered by this specification he desires and which of the paints he wishes applied. (See Note 1 on page 2163 of the above mentioned specification.) It is believed that the most effective method of cleaning is by sandblast and that the most satisfactory paint for the interior of the standpipe or elevated tank is the ready mixed red lead paint, as therein specified.]

# SPECIFICATIONS FOR PAINTING STANDPIPES

(Adopted by the New England Water Works Association at Its 1930 Annual Convention in Atlantic City. Reprinted by Permission)

The contractor shall furnish all labor, machinery, tools, rigging, brushes and all cleaning and painting materials necessary to clean and paint the standpipe in strict conformity with the requirements of the specifications.

*Cleaning.* After the standpipe is erected and all leaks stopped, it shall be thoroughly cleaned by an acceptable method.

*Sandblast Method.* The sandblast equipment shall be in good working condition and shall have ample capacity to furnish the required volume of compressed air at sufficient pressure to operate the sandblast effectively. The sand used shall be largely composed of silica grains and shall be sharp and as coarse as it is practicable to use.

The operator of the sandblast nozzle shall be protected from dust by a suitable helmet.

All rust, mill scale, grease, or other material shall be removed from the steel so as to expose the bright metal over the entire surface to be cleaned.

No more area shall be sandblasted than can be painted on the same day that the cleaning is done.

If rust forms on the cleaned surface, from any cause, it shall be removed by the sandblast before any paint is applied.

*Motor-driven wire brushes.* Motor-driven wire brushes shall have ample power and shall be operated at a speed and pressure which will clean the steel thoroughly. Brushes which become badly worn shall be replaced by new ones.

*Scrapers and wire brushes operated by hand.* All dirt, rust and loose mill scale shall be removed by means of scrapers and wire brushes.

If any rust forms on the cleaned surface it shall be entirely removed before any paint is applied.

*Brushes.* Brushes shall be new and of superior quality, having strong and elastic bristles; the brushes shall be of the proper size and shape to admit of thoroughly brushing out the paint without excessive exertion.

*Quality of materials and provision for tests.* All material to be used in painting the standpipe, for which either standard or tentative specifications have been formulated by the American Society for Testing Materials, shall conform to the latest publication of such specifications.

Litharge shall be pure lead monoxide (PbO) and shall be ground very fine.

Drier shall be of superior quality free from benzine and resin. It shall be made by a manufacturer of established reputation.

All materials, so far as practicable, shall be delivered at the site of the work in original containers showing the brand and manufacturer's name. Sufficient time shall be allowed between the delivery of this material and the beginning of work to collect and analyze samples. No paint shall be applied to the standpipe until approved by the authorized representative of the Owner.

*Application of paint.* The standpipe shall be given three coats of paint on both outside and inside surfaces. The first coat of paint shall be applied to the cleaned and perfectly dry surface before rusting begins, the second and third coats shall be applied only under favorable conditions. All paint shall be applied by skilled painters and shall be carefully brushed out to a thin, uniform coating, working carefully around rivet heads, joints and other irregularities in the surface. Each coat shall be allowed to dry thoroughly before the next coat is applied.

The paint shall not be applied during wet or foggy weather, or when the temperature of the air is below 50°F.

*Note 1.* Stipulations applying to three methods of cleaning standpipes have been included in the specifications. The method best suited to the case in hand should be selected and the corresponding requirements should be incorporated in the specifications. The kind of paint to be used and, unless it is a commercial ready-mixed product, its composition should be stipulated. Paint sprayers may be used for outside painting, but the large proportion of pigment to vehicle in the inside paint makes its application by machines difficult.

*Note 2. Repainting standpipes.* If the old paint is in bad condition the tank should be cleaned and painted in accordance with the specifications for new work so far as applicable. If a large proportion of the old paint is in fair condition it may be sufficient to clean the surface with scrapers and wire brushes, touch up bare places and apply two coats of paint instead of three.

*Paint formulas for standpipes*

*Inside first coat:*

| | |
|---|---|
| Paste red lead | 100 lb. |
| Boiled linseed oil | 1⅝ gal. |
| Fine litharge | 10 lb. |

Beat up the litharge in one quart of boiled oil and add to paint. Makes about 4.25 gal. of paint

*Inside second coat:*

Same as first with addition of ¾ lb. of paste lampblack to each batch.

*Inside third coat:*

Same as first except as follows: Add 1½ lb. of paste lampblack to each batch.

*Outside first coat:*

| | |
|---|---|
| Paste red lead | 100 lb. |
| Raw linseed oil | 2⅛ gal. |
| Drier | 1 qt. |
| Turpentine | 1 qt. |

Makes about 4.87 gal. of paint

*Outside second coat:*

| | |
|---|---|
| Paste white lead | 100 lb. |
| Raw linseed oil | 3¼ gal. |
| Turpentine | 1 qt. |
| Drier | 1 pt. |
| Paste lampblack | 2 oz. |
| Paste French ochre | 4 oz. |

Makes about 6.00 gal. of paint

*Outside third coat:*

| | |
|---|---|
| Paste white lead | 100 lb. |
| Raw linseed oil | 3½ gal. |
| Turpentine | 1 pt. |
| Drier | 1 pt. |
| Paste lampblack | 1 oz. |
| Paste French ochre | 2 oz. |

Makes about 6.00 gal. of paint

*Note.* The above red-lead paint formulas are based on paste red lead, made by grinding 100 lb. of dry red lead with 8 lb. of linseed oil and heavy paste white lead.

If it is necessary to thin the paint on account of cold weather, variation in materials, or to prevent a high gloss, pure turpentine should be added to the under-coat mixtures, but not over ½ pint to a gallon of paint, for the first and second coats, and none at all in the final inside or outside coat. Use a little more oil instead. Paint for inside coats should be as thick as it is practicable to brush out to a thin film.

Stir lampblack or other paste pigments with a small amount of paint until fluid, before adding to the main body of paint.

Paint containing litharge should be used before it begins to thicken from the action of the litharge on the oil.

It is of the utmost importance to allow sufficient time between coats for the paint film to dry thoroughly. The last coat on the inside should be allowed to dry at least one week, under favorable conditions, and longer in cold weather, before filling the tank.

### READY-MIXED PAINTS

*Composition of paints as given by a leading manufacturer of lead products*

*Inside of standpipe:*

| | | *Percent by weight* |
|---|---|---|
| First coat, color orange-red, weight 28 lb. per gal. | Red lead | 79.69 |
| | Raw linseed oil | 19.31 |
| | Liquid drier | 1.00 |
| 100 lb. = 3.57 gal. paint | | |
| Second coat, color light brown, weight 26.75 lb. per gal. | Red lead | 79.05 |
| | Lampblack | 0.35 |
| | Raw linseed oil | 19.75 |
| | Liquid drier | 0.85 |
| Third coat, color dark brown, weight 25.19 lb. per gal. | Red lead | 76.23 |
| | Lampblack | 1.40 |
| | Raw linseed oil | 21.34 |
| | Liquid drier | 1.03 |
| 100 lb. = 3.97 gal. paint | | |

Add to the first and second coats 2 lb. of litharge, mixed with ½ pint of pure turpentine for each gallon of paint.

Add to the third coat 2¼ lb. of litharge mixed with ½ pint of raw linseed oil.

All paint to which litharge has been added should be used before the paint begins to thicken.

*Outside of standpipe.* For the *outside*, first coat, use same paint as the *inside* first coat, without addition of litharge.

The second and third coats may consist of any good ready-mixed paint of the color specified by the owner.

### USE OF DRY RED LEAD IN MAKING PAINT

Paint made with dry instead of paste red lead is subject to the following disadvantages. The contents of unaltered litharge in the commercial pigment may vary between 2 and 20 percent, making it difficult to determine the proper amount of litharge, if any, to add

for the inner coats. The red lead is generally coarser than that used in paste to start with and is not ground in oil by paint machines in the process of manufacture. For this reason it requires almost constant stirring during application in order to obtain a uniform coating and does not work so well under the brush.

It is more difficult to mix the dry pigment than the paste with oil and there is some danger of injurious effect on the health of the workmen in using the dry pigment.

When for any reason it is necessary to use a paint made with dry red lead and oil, the proportions for under-water exposure should be 100 lb. of dry red lead and 3 gal. of boiled linseed oil, together with about 10 lb. of litharge if the red lead contains 2 percent of litharge, or less than 10 lb. if the proportion of litharge in the red lead is larger. The additional litharge should be mixed with sufficient oil to make a thin paste before being added to the paint. If the pigment contains more than 10 percent of litharge, no litharge need be added.

The second and third coats should be the same as the first coat with the addition of lampblack, as in the paste red lead formulas.

The outside priming coat should be the same as the inside first coat except that the litharge is omitted.

### HOT COATINGS

Hot coatings are ordinarily applied to the inside of the standpipe only. They generally consist of a priming coat applied cold and a second applied hot, both made of bituminous materials.

When hot coatings are employed, it is of vital importance to have the cleaning of the steel done in the most thorough manner, so that the bare metal is exposed over the entire surface to be coated.

The coatings are generally applied by the manufacturers or their representatives, and the work should be done by their own methods if they are to be held responsible for the results.

The outside painting is generally done under the same contract, using ordinary painting materials. The specifications and formulas previously given for outside painting are applicable to this part of the work, or the contractor may be allowed to use paints which he is willing to guarantee in a satisfactory manner.

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## DISCUSSION

ALFRED O. DOANE:[1] At the request of Mr. L. R. Howson, I am sending you the following comments on the revised standard specifications for steel standpipes and elevated tanks.

I approve the use of standard specifications of the American Society for Testing Materials, A-7 structural steel for bridges, for plates and rolled shapes, but think it would be advisable to stipulate, at least for important work, that the plates shall be ordered by thickness, (see Section 11, Paragraph C of the Specifications), as otherwise the contractor will probably order them by weight per square foot as this will be cheaper for him. On account of the spring of the rolls, steel plates are a little thicker in the middle than on the edges, and if ordered by weight per square foot, they are liable to be materially thinner than the nominal thickness on the edges, and as the riveted joints will be made in this portion of the plate, the effect of the deficiency in thickness will be more serious than if it occurred in the body of the plates.

If ordered by thickness, it is provided that "The thickness of each plate shall not vary more than 0.01 inch under that ordered." It is obvious that this will have a favorable effect on the strength of the riveted joint.

In Section 26 of the elevated tank and section 55 of the standpipe specifications, "No scarfing shall be done at a black heat." I find that "blue heat" is more commonly used to describe this temperature at which it is injurious to work the steel than "black heat" which is used by some of our local boiler makers.

I am gratified to learn that the Committee has adopted the New England Water Works Association specifications for painting standpipes. Our practice on the Metropolitan Water Works has been to sand blast new work and use paste red lead as a basis for the interior paint. This gives somewhat more flexibility in the proportioning of the paint for existing condition.

Under experienced and careful supervision it works well, but for most work it is probable that ready mixed paint made by a reputable manufacturer would be preferable.

[1] Senior Mechanical Engineer, Water Division, Metropolitan District Commission, Boston, Mass.

GEORGE T. HORTON:[2] My ideas on the standard specification for steel standpipes and elevated tanks as prepared by your Committee are about as follows. The specification as a whole is excellent, particularly its intent. My criticisms are mainly directed against clauses which to my mind are too narrowly drawn.

In Section 6 reference is made to a horizontal circular girder connecting the supporting columns. Very often this girder is not circular, but polygonal.

The sense of Section 8, referring to balcony, could well be embodied in the clauses referring to the horizontal girder.

In Section 9, I would omit the second and third paragraphs referring to special cast steel flanges, as this type of connection is very seldom used and only for special purposes.

Section 11 might be either condensed or amplified. Overflows are constructed both inside and outside, and an inside overflow is not visible.

There is a discrepancy between the note in Section 9 in reference to the amount of load carried on the riser and that described under live load requirements in Section 15.

I suggest the following clause be added to Section 17:

"Careful analysis and determination of the stresses, both tension and compression, that may exist in a suspended tank bottom, must be made and provided for under the Unit Stresses set forth in Section 16."

Under Section 20 I would suggest the following clause:

Five-eighths inch rivets and under may be driven cold. When so driven the heads may be driven practically flat."

In Section 24 I would omit the angle of the bevel shearing, it being the contractor's duty to make his work tight. Such requirements have no virtue; in fact the entire requirement of beveling might as well be omitted as it is not in reality necessary and is only a trade practice followed in the tank industry. It is not done in ship construction nor, I believe, in heavy boiler construction.

In Section 26, if the clause relative to the temperature of plates when scarfing is to be taken seriously, it would probably be better to require all scarfs to be planed. This is a universal custom abroad, assuring excellent workmanship and appearance, and is one I would be glad to see introduced into this country.

Section 29. The procedure described should apply only to those

[2] President, Chicago Bridge and Iron Works, Chicago, Ill.

cases where the foundations are out of level. In cases where the foundations are well constructed it is not necessary.

Section 33. Some amelioration of the statement that leaky rivets must be cut out and replaced by new ones, should be introduced. I think I once recommended the statement: "Minute leaks in tight rivets may be caulked," this being an attempt to recognize in the specifications what all intelligent foremen and inspectors do in fact.

STANDPIPE SPECIFICATIONS

Section 36. Perhaps a clause should be inserted defining the design of the foundation proper in order to prevent overturning.

Suggestions made relative to elevated tank design may pertain to standpipe design where applicable.

I hope that your Committee will soon conclude your recommendations and that the specification for the Association will shortly thereafter be available.

J. O. JACKSON:[3] Some consideration should be given to the following comments, some of which we think are quite important.

*4. Form and proportions*

On page 3, the last paragraph of the above section specifies that "the total height of an elevated tank measured from the lowest point of the bottom of the overflow level should not be more than $1\frac{1}{2}$ times the diameter of the tank, except in unusual cases."

Our standard hemispherical bottom tanks are as follows:

| CAPACITY OF TANK | DIAMETER OF TANK | OVERALL HEIGHT | | 1.5 D | |
|---|---|---|---|---|---|
| *gallons* | *feet* | *feet* | *inches* | *feet* | *inches* |
| 20,000 | 13 | 22 | 5 | 19 | 6 |
| 30,000 | 15 | 25 | $9\frac{1}{2}$ | 22 | 6 |
| 40,000 | 17 | 26 | $\frac{1}{2}$ | 25 | 6 |
| 60,000 | 19 | 32 | $\frac{3}{8}$ | 28 | 6 |
| 75,000 | 21 | 33 | $\frac{3}{8}$ | 31 | 6 |

We have found these proportions to be economical and believe that your specification would be an unnecessary burden on the pur-

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chaser of a small elevated tank where small range of head was of no particular value. We would recommend that the ratio be changed to 1.75 times the diameter of the tank instead of 1.5 times the diameter of the tank.

*6. Towers*

On page 3, fourth paragraph, indicates that allowable working stresses for the flanges, if eccentric, shall be reduced 25 percent. We see no reason for this reduction for if the flanges are properly supported laterally, there is no reason for them not to work at capacity. We believe this is an unfair specification both for the purchaser and fabricator. We would recommend that it be revised to read that if eccentric the flanges shall be properly supported in a lateral direction.

In the latter part of this same paragraph, it is stated that the horizontal component of the maximum column load shall be transmitted to the girder without subjecting the rivets to tension. There are cases where some tension in the rivet heads cannot be avoided and in order to make the specifications usable, we would recommend that this clause be omitted.

*7. Diagonal bracing in the tower*

The second paragraph of this section states that the bracing in the tower shall be calculated for an initial stress of 3,000 pounds per square inch. We have recently made calculations which have been substantiated by laboratory experiment and which prove that the initial tension in tower rods does not act to increase the load in the rods when they are carrying a full wind load. We would, therefore, recommend that this statement be modified to read that the diagonal bracing in the tower shall be adjusted so that an initial stress of 3,000 pounds per square inch is put into the rods at the time of erection.

We believe it is not wise to specify that on tanks of 500,000 gallons capacity and over that the sway bracing shall be by riveted members. We have had several cases in the past where riveted sway bracing has not proved entirely satisfactory on large tanks and we believe this is a detail of design which should be left open.

*8. Balcony*

In order to conform with present standards, we would recommend that the balcony width be reduced to 18 inches for tanks up to 15

feet in diameter and not less than 24 inches for tanks of larger diameter. We have found balconies of this size to be adequate and consider it unwise to burden the purchaser with additional expense for a wider balcony when one of the width recommended above is adequate.

We would also recommend that the thickness of the balcony floor plates be a minimum of $\frac{1}{4}$ instead of $\frac{5}{16}$ inch as in the case of small structures the $\frac{1}{4}$ inch balcony plate is adequate.

*9. Riser pipe*

We would recommend that the thickness of the flanged bottom plates for the riser be reduced to $\frac{1}{4}$ inch as in the case of small tanks, this thickness is sufficient. In the second paragraph it is our standard practice to weld a steel flanged piece to the base of the riser which construction we consider superior to a cast steel piece riveted to the bottom plate and would recommend that the specification be left open in this connection.

The fourth paragraph of this section requires that a crane be provided to hold the cover of the manhole. We would recommend that this requirement be omitted as it is not practical to use a crane with a standard oval manhole which must be inserted to the inside of the riser and then brought to bear against the gasket on the inside by means of manhole arches. Perhaps the word "chain" was intended. This is not necessary, however, because the manhole cover is never removed unless the riser is empty and in that case the cover is only 2 feet 6 inches from the bottom of the riser and may be easily reached from the outside.

*10. Top stiffeners*

We would recommend that the minimum thickness of $\frac{3}{8}$ inch for any structural shape making up the stiffener be reduced to $\frac{1}{4}$ inch or omitted entirely as a suitable stiffener may often be built up of $\frac{1}{4}$ inch material and in some cases channel stiffeners are used, the webs of which are less than $\frac{1}{4}$ inch.

*13. Ladders*

We would recommend reducing the size of the side members of the ladder to $\frac{5}{16}$ by 2 inches as in the case of small tanks this is adequate. We would also recommend changing the size of the rungs to $\frac{5}{8}$ inch round or square as some manufacturers use round rungs as standard.

*16. Allowable stresses*

The shearing stresses are specified for rivets as 11,250 pounds for rivets in cylindrical portion of tank, 9,000 pounds for rivets in bottom course and formed bottom plates and 7,500 pounds for rivets in main column connection to tank and in all cases it is indicated that the net section is to be used. We would recommend that the expression "net section" be changed to "upset section" as the actual diameter of the rivet is the upset diameter and if the upset diameter is not used in the calculations an unbalanced riveted joint results which is uneconomical and does not ensure a better job.

We would recommend that the shearing stress in the rivets in main column connection to tank be increased from 7,500 pounds per square inch upset section to 9,000 pounds as there is not justification for such a conservative figure as 7,500 pounds. We have had many cases where it would be impossible to get enough rivets in the column tank connection using the 7,500 pounds figure and if this is allowed to be used in the specification, we feel certain that it will not be generally followed.

*19. Minimum thickness*

We would recommend $\frac{5}{16}$ inch minimum thickness for tanks over 500,000 gallons instead of $\frac{3}{8}$ inch. Five-sixteenth inch represents an increase of 25 per cent over $\frac{1}{4}$ inch and if a $\frac{3}{8}$ inch minimum is used, the cost of the structure to the customer is unnecessarily and unjustifiably increased.

In the last paragraph of this section, we would recommend changing the minimum thickness of cover plates to be not less than one-fiftieth of the distance between rivet lines instead of one-fortieth as the latter figures prevent the use of $\frac{1}{4}$-inch covers on 10- and 12-inch channels which is fairly standard practice.

*20. Rivets*

We would recommend that the minimum permissible diameter of rivets be reduced to $\frac{7}{16}$ inch or omitted entirely as in the case of very small tanks, it is economical to drive cold rivets $\frac{7}{16}$ or $\frac{1}{2}$ inch in diameter.

*21. Spacing of rivets*

With reference to the third paragraph of this section, we would recommend that this requirement be omitted entirely as this limita-

tion unwisely limits the efficiency of butt joints by making it necessary to place the rivets in the outer row a maximum of thirty times the thickness of the thinnest plate. We think this limitation is unwise and it should be omitted.

*22. Edge distance*

This first paragraph requires that the distance between the center of the outer row of rivets and the edge of the plates shall not be less than one and one-half times the diameter of the rivets or where edges are caulked, shall not exceed one and three-fourth times the diameter of the rivets. It is not stated whether the distance is to be measured to the toe or to the heel of the bevel although the fourth paragraph of section 25 indicates that the plates are to be laid out to the heel of the bevel. We would recommend that it be specified in section 22 that for caulked edges the toe of the bevel shall be not more than one and three-quarters times the diameter of the rivets and the heel of the bevel not less than one and one-half times the diameter of the rivets from the center line or gauge line of the rivets.

*23. Shearing*

The specifications do not provide for plates over $\frac{1}{2}$ inch thick, up to and including $\frac{5}{8}$ inch thick. It is recommended that this paragraph be made the same as No. 52 for standpipes. It is also recommended that the last sentence be modified as follows: "Plates over $\frac{5}{8}$-inch thick shall have caulking edges planed or flame cut by automatic machine."

This is a recent development in beveling plates and produces a bevelled edge which is superior to a bevel sheared edge and is equivalent to a planed edge. If there is any question about this point, samples of this work will be sent for your inspection. This process is being used by several companies on plates up to 2 inches in thickness.

*26. Shaping plates and angles*

We consider that the most satisfactory scarfing is that done by extremely high pressure with the plates cold. These specifications would apparently not permit such scarfing and we recommend that they be revised as follows: "Plates to be scarfed may be heated to a cherry red color, but not hot enough to ignite a piece of hard wood when applied to it or if scarfed by pressure, such scarfing may be done cold, etc."

*37. Pipe connections*

We recommend that as in the case of the tank and tower, the specifications be left open for the use of a steel flanged piece as well as a cast steel flanged piece and that it be left optional whether the piece be riveted or welded to the standpipe bottom.

*38. Manhole*

The same comments in connection with the crane apply as in the tank and tower specifications.

The second paragraph of this section requires that the contractor shall grout between the steel plates and the concrete after the standpipe is erected. This is more costly and we believe generally less satisfactory than lowering the tank bottom on a sand and cement cushion as the latter can be distributed more uniformly and leaves fewer voids than in the case of grout pumped into grout holes in the tank bottom. We recommend that the specifications be changed by adding "—after the standpipe is erected, or by lowering the tank bottom on a 2-inch cushion of cement and sand mixed dry in the proportions of one part of cement to three parts of sand and spread uniformly to the above thickness over the concrete foundations."

*39. Top stiffeners*

We recommend that the minimum thickness of $\frac{3}{8}$ inch be reduced to $\frac{1}{4}$ inch as recommended for tanks and towers.

*42. Ladders*

It is our opinion that it is not practical to provide a ladder "extending from a point 12 feet above the top of the concrete foundation to the peak of the roof, so constructed and fastened to the finial, etc., etc." We do not believe that in the case of a standpipe, say 100 feet high, that a ladder of this length would function properly. We would suggest that the specifications be revised to require a revolving ladder on the roof with provision for attaching boatswain's chair to the end of the revolving ladder from which all portions of the standpipe shell could be reached. We would also recommend including a fixed ladder on the side of the standpipe.

*45. Allowable stresses*

With reference to shearing and bearing stresses for rivets, we would

recommend that these apply to the upset sections instead of the net sections for the reasons stated in connection with tanks and towers.

*48. Minimum thickness*

We believe the minimum thickness of $\frac{3}{8}$ inch is not justified for standpipes over 500,000 gallons capacity. One of our standard designs for a 500,000 gallon standpipe has no metal in the standpipe thicker than $\frac{5}{16}$-inch. In special cases where there are unusual corrosion conditions, heavier metal may be specified, but this large minimum places too much of a burden on the purchaser and in our opinion will simply result in the specification not being widely followed.

With reference to the second paragraph of this section, we consider that $\frac{1}{2}$-inch thickness of bottom plates is excessive in some cases for standpipes 175 feet high and would suggest that the maximum be reduced to $\frac{3}{8}$ inch.

*50. Spacing rivets*

The same comments apply as in connection with tanks and towers.

*51. Edge distance*

The same comments apply as in connection with tanks and towers.

*59. Placing bottom of standpipe on foundation*

The last line of the second paragraph requires that the bottom shall be absolutely level. We recommend that this be modified as it is impossible to make a bottom absolutely level and we do not believe anyone could tell whether a bottom was absolutely level even if it were possible. We would suggest that the sentence be revised to read, "The bottom shall be set so that all points in the circumference of the bottom angle are within $\frac{1}{8}$ inch plus or minus of the same level plane."

The first paragraph of this section states that the bottom shall be supported and lowered upon the foundation in such a manner as to prevent sag. We would be interested in knowing how this could be done, but we would recommend that the paragraph be omitted as unless every square inch of the bottom is supported during the lowering process there is certain to be some sag and we know of no objection to the sag anyway, so why make the specifications impractical.

The above points cover our criticisms and recommendations and we earnestly hope that you will see fit to embody them in the final draft of the specifications. If this is done, we can assure you that as far as we are concerned, we shall endorse them most heartily as we believe that, aside from the above points, they are very practical and are a decided improvement over any existing specifications.

LYLE MERRICK:[4] I have carefully read the tentative draft of the American Water Works Association's Specifications for Steel Standpipes and Elevated Tanks and offer the following comments:

(1) Section 6, In paragraph 5, I recommend that the last of the paragraph be changed to read: "or else shall be a structural shape connected to the tank by rivets above and adjacent to the joint between the shell and bottom."

In my opinion it is not good practice to place a stiffener over a riveted seam because it hinders the driving of tight rivets by increasing the grip, covering up an edge of a caulked plate and rendering it more difficult to get at the rivet to drive it.

(2) In Section 9, paragraph 2 and Section 37 page 17 it might be well to state that the cast steel flange piece is to be riveted to the underside of the bottom.

(3) In Sections 9 and 38, the sizes of the manways are given as 12 by 16 inches and 18 by 24 inches. These are special with many supply houses and it would help delivery and prices to change these to 11 by 15 inches and 18 by 22 inches for minimum sizes.

(4) In Section 42 under "Ladder" it is unusual and perhaps hazardous to install a revolving ladder which extends from the peak of the roof to within 12 feet of the ground. I recommend that only the portion of the ladder on the roof be revolving and a painter's trolley and track be provided on the outside of the shell near the top.

(5) I believe that the fifth paragraph of Section 54 should be eliminated because it will increase fabricating cost unnecessarily in view of requirement contained in the sixth paragraph on the same page in regard to removing burrs from rivet holes.

(6) Under "Information to be furnished by Purchaser" it is also essential that the Purchaser advise definitely regarding source and quantity of water available for testing.

(7) I do not have a copy of the specification on painting as adopted

[4] Chief Engineer, The Petroleum Iron Works Company, Sharon, Pa.

by the New England Water Works Association mentioned in these American Water Works Association Specifications and cannot comment on them at this time.

The question of painting is a very important one, but I find very little agreement among purchasers as to kind of paint and method of application.

(8) The members of the committee are to be congratulated on the excellent specifications for standpipes and elevated tanks which they have prepared.

These specifications should go a long way toward producing first class water storage tanks for the country at a reasonable cost.

H. A. SWEET:[5] We have received copy of the final draft of Standard Specifications for Steel Standpipes and Elevated Tanks, as prepared by Sub-Committee 7-H of the American Water Works Association, together with letter of February 10 from Mr. L. R. Howson, Chairman. Our former comments contained in letters of September 22, 1930, January 15–19, 1931 have undoubtedly been given consideration by your committee in preparing the final draft, and we therefore are not repeating any of the comments at this time wherein the final draft differs from our suggestions. There are a few features, however, which we would suggest be given additional consideration for final publication of your rules.

Referring to Sections 9 and 37, a "Silt Stop" is required which must set 6 inches high above the bottom of the standpipe or large riser and should be suitable for setting into the flanged piece. It is not clear to us just what this "Silt Stop" would be if in addition to the flanged fitting and discharge pipe. For standpipes, the end of the discharge pipe or the fitting for caulking provides the necessary 6-inch settling basin. For elevated tanks with large risers, a deeper settling basin is needed, particularly for your tanks where constant circulation is taking place, and we would suggest a depth of at least 3 feet for the settling basin, which could be obtained by extending the discharge pipe through the flanged fitting this amount. We are inclined to think that an inside blow-off valve would be advisable for elevated tanks for municipal service to permit removing the sediment occasionally, merely by opening the valve without necessitating draining the tank and removing the manhole cover. The larger

[5] Engineer, Factory Mutual Laboratories, Boston, Mass.

tank companies have standard approved designs for inside blow-off valves, operated by a wrench on a valve stem extending through a stuffing box near the base of the large riser.

Referring to the last sentence, paragraph 4, Section 9, it would be well to state for what diameter of large riser the 18- by 24-inch manhole should be used.

Referring to Section 14, it would be well to specify the type of metal to be used for diagonal wind rods. The term "mild steel" is rather indefinite and we believe it would be well to use one of the designations of the American Society for Testing Materials. We would suggest such materials as A-84 engine bolt iron, or A-7 rivet steel.

Referring to paragraph 3, Section 49, we believe that other materials than copper bearing rivets could be used for the roof and balcony. The term "or other material resistant to air corrosion" specified in Section 43, could be used in Section 49 in reference to the rivets, as we believe that genuine wrought iron or Toncan metal would be as satisfactory as the copper bearing steel.

Section 59 requires that the bottom of standpipes be supported and lowered upon the foundation furnished by the purchaser in such a manner as to prevent sag. It is not clear how the bottom could be lowered so as to prevent sagging. The bottom is usually built on horses and those near the middle removed before lowering onto the foundation, and the bottom usually sags 2 feet or more during this process. The specification requires that the bottom plate be absolutely level, but we have had a few instances of where the standpipe as finished has not been exactly plumb, which may have partly been due to not plumbing the plates as they were erected. In addition to making the bottom plate level, it would be well to have the vertical plates plumb. This item is covered by Section 609 in the Factory Mutual Specifications, which we sent you previously.

It would be well to locate the horizontal girder at the top of inclined columns at the joint between the cylindrical plate and the suspended bottom. This feature does not seem to be covered in paragraph 3, Section 6. This feature is covered in article 1219 in the Factory Mutual booklet by the first sentence.

Referring to paragraph 4, Section 6, the bending moment in the circular girder is usually considered as carried by the flanges, while the shear is carried by the web plate, and the compression or thrust

due to arch action of the girder is carried by the flanges and web in proportion to their areas at the section considered. It would seem as if the sentence in your rules while intending to cover the bending moment, really only covers the thrust or compression due to the arch action of the girder. This feature is covered in article 1223 of the Factory Mutual booklet.

# ABSTRACTS OF WATER WORKS LITERATURE[1]

FRANK HANNAN

**Key**: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the Journal.

**Progress in Preventing Pollution from Industrial Wastes in Wisconsin.** Eng. News-Rec., 104: 475 March 20, 1930. Encouraging results are being obtained from studies in Wisconsin on prevention of water pollution by various kinds of trade wastes. Experiments at De Forest have shown that milk wastes can be rendered unobjectionable at a reasonable cost by treatment on broken stone trickling filters. The stone should be ½ to 1 inch in size, have an effective depth of about 8 feet, and surface area sufficient to keep the dosage rate down to 2 m.g.d. per acre. Good results can be obtained with lath filters. Activated sludge treatment on a practical scale was not satisfactory. Waste treatment plants of the chemical precipitation type have been installed at 15 pea canneries. Studies have been made of the screening—soil absorption method of cannery waste disposal, with chlorination for odor control. Results indicate limited application for this disposal method. Studies on sewage treatment at Fond du Lac indicate that effective results can be secured with sewage mixed with equal volume of tannery wastes which have received preliminary sedimentation.—*R. E. Thompson.*

**Construction Methods on Small Power Plant at Difficult Site.** H. L. MAHAFFY. Cont. Rec. and Eng. Rev., 44: 361–5, March 26, 1930. Illustrated description of construction of power plant at Burrough's Falls on Nigger River in Quebec, the latest addition to system of Southern Canada Power Company. Dam is a full gravity section combining spillway and two sluiceways, and is flanked on west end by tapering gravity non-overflow section backed by earth fill and on the east end by the intake adjoining the sluices and an earth dam with concrete corewall to solid rock.—*R. E. Thompson.*

**Power Plant at Burroughs Falls.** J. R. DESLOOVER. Cont. Rec. and Eng. Rev., 44: 441–4, April 16, 1930. (Cf. previous abstract.) Description of 2000-horsepower plant of Southern Canada Power Co., placed in commission in January. Dam has maximum height of 68 feet. Of a total length of 490 feet, 140 feet is accounted for by earthfill dam, 55 feet by sluiceway section and 90

[1] Vacancies on the abstracting staff occur from time to time. Members desirous of coöperating in this work are earnestly requested to communicate with the chief abstractor, Frank Hannan, 285 Willow Avenue, Toronto 8, Ontario, Canada.

feet by the spillway; concrete wing walls and intake section making up the balance.—*R. E. Thompson.*

**Small Siphon Spillway Serves City Distribution Reservoir.** J. R. GRANT. Eng. News-Rec., **104**: 726, May 1, 1930. A small siphon spillway of compact design was recently built to discharge overflow from Vancouver Heights distributing reservoir which is being constructed in units by Greater Vancouver Water District, B.C. At present the spillway is used in connection with first unit already completed, but additional units, which will have ultimate capacity of 46 m.g., will be built to utilize same spillway. Reservoir is situated in residential district about 345 feet above, and 1600 feet back from shore line of Burrard Inlet, into which spillway discharges. The siphon spillway was selected to provide discharge for the 100 second-foot overflow because it furnishes automatic regulation, together with minimum size outlet structure. Throat of siphon provides for reservoir depth of 22 feet and discharge passes down an inclined lower leg of 3 by ½-foot cross section into stilling well, from which water flows over baffle wall into a second stilling well which connects with 24-inch outlet pipe. Latter has fall of 320 feet in length of 1600 feet, which is sufficient to take care of flow from the two 36-inch inlet pipes under their operating head. Success of siphon in maintaining water level at a pre-determined height is illustrated by reproduction of chart showing record of a week's operation.—*R. E. Thompson.*

**Gunited Column Footings Provide Continuous Reservoir Lining.** Eng. News-Rec., **104**: 698–9, April 24, 1930. In construction of small water supply reservoir in Burlingame, California, the footings below the lining, lining itself, and column pedestals were gunited in one continuous operation, resulting in continuous lining over the whole bottom of reservoir uninterrupted by supporting columns. This proved to be an economical construction procedure. Reservoir is now in use and the resulting water-tightness has been very satisfactory.—*R. E. Thompson.*

**Constructing a Pipe Line in Corrosive Soil.** J. B. LIPPINCOTT and K. Q. VOLK. Eng. News-Rec., **104**: 847–50, May 22, 1930. Detailed description of construction of 11.6-mile water supply conduit for Laguna Beach County Water District, California. Former supply by private company, from wells in small canyons back of the town, owing to recent dry years in Southern California practically failed and it was therefore necessary to seek new source of supply. The Laguna Beach County Water District, 1450 acres in area, was formed and joined hands with Irvine Ranch Company, owner of large tract of coastal land between Laguna Beach and Corona Del Mar. Reliable source of supply was located, in wells situated in delta of Santa Ana River, 11.6 miles north of Laguna. Pipe-line has maximum capacity of 15 second-feet for 75 per cent of its length, reducing to 5 second-feet at end of line, over 8 miles being of 30-inch, exteriorly gunite-coated, electrically-welded steel force main and remainder, of 18- to 27-inch centrifugally spun reinforced concrete gravity main. Maximum head in force main, including water hammer, is about 250 feet and in gravity section, 125 feet. Throughout practically its whole length pipe-line

passes through sedimentary soils of marine character which are highly corrosive. Examination of 200 feet of 12-inch 14-gage steel pipe installed in Sweetwater Company system about 15 years ago, on which ½ inch of gunite had been applied over asphalt dip, using chicken wire reinforcement, had shown that steel remained bright and gunits in good condition. Tests made to determine design characteristics for cement-coated pipe showed that, to prevent cracking, plate thickness was of greater importance than thickness of gunite coating. It was decided to use $\frac{3}{16}$-inch plate with electrically-welded girth and longitudinal seams, covered externally with ¾-inch of 1:5 gunite, reinforced with electrically-welded 14-gage wire fabric of 2 by 4-inch mesh. Experiments demonstrated importance of proper curing and necessity of bracing pipe interiorly to true circular profile until gunite had cured, after which braces could be removed without damage to coating. Experiments were also conducted on centrifugally spun pipe to determine proper mix, type of reinforcement, and time and rate of spinning. The steel pipe was made in sections 16 feet long, two 16-foot plates being used for each, the two longitudinal seams being automatically electrically butt-welded. After testing, pipe was preheated and vertically dipped in asphaltic solution. Gunite was applied to within 12 inches of each end. After 2 hours, pipe was covered with burlap and kept continuously wet during daylight hours as long as it remained in yard. No pipe was allowed to be removed until it had cured at least 10 days and all pipe was laid within 48 hours of its removal from yard. Trench was first coated with 4 inches of beach or fine dune sand, and, after pipe was laid, backfilled with sand to center of pipe. Girth welds were electrically lap-welded in field. Line was bulkheaded and tested for leakage in sections of about 1000 feet, joints then being painted with a priming coat of asphaltum, followed by heavy coat of asphaltic pipe dip, covered with reinforcing mesh, and gunited. Where line crosses Newport Bay, 24-inch flexible cast iron pipe was employed. Method of manufacturing the centrifugal pipe is described. It was steam cured for at least 12 hours before removing forms and then sprinkled for 8 days. A spun collar 8 inches in width was calked on one end of each pipe section in contractor's yard, inside diameter of which was approximately 1½ inches greater than outside diameter of pipe. Following Australian precedent, an asphaltic collar was placed in groove in end of each pipe and pipe was jacked into section previously laid until flush. Under this pressure it was intended that the plastic material should completely fill concave ends of pipe and that remaining space should be then calked with 1:1 mixture of cement and fine sand, expansion being thus provided for. It was found, however, that plastic material flowed from between ends of pipes and hung down into interior. This trouble was eliminated by using dry jute and calking with neat cement. There was then little leakage at joints on testing. All leaking joints were recalked with neat cement and line again tested, the leakage then being but 20 per cent of specified allowance of 200 gallons per inch diameter per mile per day. Line has been in service over 2 years and a recent test disclosed practically no leakage. Unusual practice is to protect steel pipe by double dipping in asphaltum and wrapping spirally with paper similar to good grade roofing paper. This has been found satisfactory where field inspections have been carefully carried out and joints properly wrapped. Wrapping is generally furnished at average price of 1.5 cents per inch

diameter. California waters, as a rule, do not cause incrustation, or tuberculation; but the soils are corrosive.—*R. E. Thompson.*

**High-Pressure Gas Line Laid Across Hudson River in Fast Time by Unusual Methods.** H. C. SANDBECK. Eng. News-Rec., 104: 810–13, May 15, 1930. Illustrated description of laying of 6520 feet of 8-inch high-pressure gas line across Hudson River between Beacon and Newburgh, N. Y., in 11 days and 7 hours. The pipe, assembled and lowered from car float 280 feet long, was kept to proper curvature, avoiding undue stresses, by means of pontoons attached to pipe and later released as bottom was reached. Width of river at pipe crossing is 6300 feet, and depth varies between 20 and 50 feet. Pipe used was of extra strong steel, weighing 43 pounds per foot, delivered in 40-foot lengths. Lengths were welded together in pairs, joints tested, and specified coating applied prior to laying operation. Joints were butt-welded throughout and each joint reinforced by short sleeve of 9-inch standard pipe fitting over it and welded at both ends to the pipe. Important advantages of this design are that 2 out of 3 welds must be faulty before the pipe can leak and that factor of safety against excessive laying stresses is considerably greater at joint than along pipe itself. The greatest benefit is the ease of testing the joints. Each 9-inch sleeve had a tap for ¼-inch pipe to which, for test, were attached a pressure gage and a valve connection to a cylinder of nitrogen under pressure of about 350 pounds. The $\frac{5}{32}$-inch annular space between pipe and sleeve had a volume of about 0.1 cubic foot. Leakage from outside joints could now be detected with soap suds and from inside joint by drop in pressure. All pipe was given one coat of asphaltic primer solution and one coat of asphaltic enamel, a touch-up coat being applied as pipe was fed out. As a result of experience with 4-inch gas line laid at the same point in 1925 which developed pitting in one section due to electrolysis, an insulation joint was placed on each side of river 15 feet out from high-water line on the bank. Furthermore, pipe was incased in concrete to a distance of 100 feet from same water-line. The electrolysis of the other line was probably caused by stray currents from an electric railway and was checked by installation of insulating joints in 1927. The total cost was $38,181.44.—*R. E. Thompson.*

**Pipe and Joints in Jerusalem.** ANDREW KOCH. Eng. News-Rec., 104: 864–5, May 22, 1930. About 93 miles of water mains have been laid in Jerusalem's new water works since 1921. Pipe is of 3 kinds, maximum size being 8 inches. Most of pipe, 4 to 6 inches in diameter, is of wrought iron with thread-and-socket joints, protected with (1) Dr. ANGUS SMITHS' coating and (2) after laying, with 3 coats of red lead. No serious defects have been found in this pipe. Four years ago 6.2 miles of 8-inch wrought iron pipe with victaulic joints were laid, part of which is in pressure line under 284 pounds per square inch. No defect, leakage, or corrosion has been experienced with this line even during the earthquake, which at certain places moved pressure main several inches from its original position. A section of Mannesmann pipe with spigot-and-socket joints has been found less satisfactory than the pipe with victaulic joints.—*R. E. Thompson.*

**Heavy Rains in New Orleans Tax Drainage Facilities.** GEORGE G. EARL. Eng. News-Rec., 104: 907, May 29, 1930. During storm of September 6–7, 1929, a total of 10.75 inches of rain fell in about 13 hours in business district of New Orleans and 12.96 inches in uptown area. The intensities were: 3.18 inches in 1 hour; 5.71 inches in 2 hours; 8.29 inches in 5 hours; and 8.84 inches in 8 hours. Four or five heavier rains have been recorded in the past, the heaviest on record being 14 inches in 9 hours.—*R. E. Thompson.*

**Sterilization of Drinking Water by the Oligodynamic Action of "Katadyn."** Gesundheits-Ingenieur, 1929: 47, 804. From Bulletin of Hygiene, 6: 2, 188, February, 1931. Water artificially infected with a variety of organisms and exposed to action of "Katadyn" materials showed rapid decrease in number of viable organisms. After a few hours, even with heavy infection, water becomes sterile. Pure water exposed to action of the material itself acquires disinfectant properties; such activated water is non-toxic to animals, even when given over a period of months.—*Arthur P. Miller.*

**Observations on the Hygienic Condition of Public Swimming Pools.** A. M. M. GRIERSON. J. Hygiene, 1930, 30: 66–78. From Bulletin of Hygiene, 6: 2, 188, February 1931. Where no filtration or disinfection is practiced in swimming pools, bacteriological examination of the water showed it to be grossly impure even after a day's use. Such organisms as diphtheroids and *B. dysenteriae* were found. In unsterilized tap water certain pathogenic organisms were found to die within 48 hours; but in sterilized tap water they survived for longer periods. Staphylococci, haemolytic streptococci, pneumococci, and *B. Typhosum* are demonstrable at 144 hours, while *C. diphtheriae* are shown at 72 hours. Meningococci and gonococci survive only 2½ hours at 22°C. Hence, very delicate pathogenic organisms can remain viable for a period sufficiently long to permit infection. The author concludes that a fill and draw system for swimming pools is inadequate and that the only safe process is continuous filtration combined with aëration and chlorination.—*Arthur P. Miller.*

**Purification of Swimming Pool Water.** H. T. CALVERT. Surveyor, 1930, 78: 49. From Bulletin of Hygiene, 6: 2, 188–189, February, 1931. Continual purification of swimming pool water is now essential. From the economic viewpoint only, installation of mechanical filtration equipment is a good proposition if compared with cost of frequently changing the water. Filters should he adequate to treat pool contents within period of four hours and coagulants are essential. The pH of pool should be 7.5 to 8.0. Filtration clarifies the water; but chlorination with a residual between 0.2 and 0.5 parts per million is absolutely necessary for sufficient bacterial removal.—*Arthur P. Miller.*

**Swimming Pools and the Public Health.** BORDAS. Ann. d'Hyg. Publ., Indust. et Sociale, 1929, 7: 709–13. From Bulletin of Hygiene, 6: 2, 189, February, 1931. The river swimming pools in Paris are described and the danger of infections from swimming pools stressed. The Paris situation demands adequate legal and administrative control.—*Arthur P. Miller.*

**The Hygiene of Swimming and Public Baths.** R. D. De La Rivière. Bull. Acad. Med., 1929, 10: 654–8. From Bulletin of Hygiene, 6: 2, 189, February, 1931. In a pool in Paris the number of organisms found was 40,000 *B. coli* and 130,000 other organisms per cubic centimeter. Infections of respiratory, intestinal, and genital systems, skin diseases, acute conjunctivitis, and infective jaundice have been contracted in swimming pools. Better constructed pools, filtration and disinfection of the water, and insistence on personal hygiene of bathers are essential.—*Arthur P. Miller.*

**Administration of Swimming Pool Standards in Detroit.** W. H. Cary, Jr. Amer. J. Pub. Health, 1930, 20: 727–33. From Bulletin of Hygiene, 6: 2, 189, February, 1931. Describes methods employed to insure hygienic conditions in swimming pools of Detroit. By system of rigid inspection, exclusion of the infected, and attention to cleanliness of floors, the number of cases of plantar warts and ringworm has been reduced.—*Arthur P. Miller.*

**A Retrospection of Chlorination. Chlorination of Water.** Morris M. Cohn. Municipal Sanitation, 2: 7, 333, July 1931. Early beginnings of water chlorination are veiled in doubt: but many pioneers of practical water chlorination still live to see its marvelous extension. In 1896, George W. Fuller applied chloride of lime to raw water entering Jewell filters in the historic Louisville experiments. Thus was pre-chlorination of water evolved. In 1902, Dr. Duyk applied ferric chloride and chlorinated lime to raw waters at Middlekirke, Belgium, for coagulation and sterilization. Two years later, Houston prechlorinated influent to slow sand filters at Lincoln, England, with chloride of lime. In 1908, Johnson treated Bubbly Creek water of Chicago stock yards with chloride of lime and copper sulphate, obtaining water of better quality than local public supply. In same year Jersey City adopted chloride of lime treatment. In 1910, Major Darnall first used liquid chlorine to sterilize the small supply of Fort Meyers, Va. In 1913, first practical chlorine control apparatus appeared. Original use of chlorine was for sterilization. First essential of drinking water is freedom from harmful bacteria. This is made possible by germicidal action of chlorine. With passage of time sources of municipal water supplies tend to become more polluted, imposing greater bacterial loads upon filters. It is significant that, for a majority of raw waters, filtration unaided by chlorination cannot be depended upon to produce effluents conforming with *B. coli* standard of U. S. Treasury Department. When chlorine is applied to water series of complex reactions is set up. It is residual chlorine, over and above amount demanded by organic and inorganic compounds present in water, which is available for bacterial destruction. Oxidizable inorganic materials include ferrous salts, nitrites, and hydrogen sulphide; organic include urea, amido- compounds, cellulose, and humus from soil washings. Since waters vary in character, amount of chlorine needed for sterilization is different for different waters, as well as for the same water at different times. Usually, satisfactory sterilization requires from 0.1 to 0.3 p.p.m. free chlorine after satisfactory contact period. Sterilization effect is directly related to chlorine concentration and inversely to concentration of oxidizable matter present. Tastes and odors resulting from application of chlorine

are of two types. "Chlorinous" odor, a pungent, sharp sensation, is direct result of free chlorine. The chloro- tastes, of iodoform character, result from reaction of chlorine with phenols, cresols, and some algae oils.—*R. E. Noble.*

**Mobile Brings Its Water Works Up To Date.** S. H. SMITH. Water Works Eng., 84: 11, 813, June 3, 1931. Old pumping equipment was replaced by modern at Bienville Station; single-stage centrifugal pump, of 9,500,000 gallons daily capacity, was directly connected to 400-h.p. synchronous motor and to 450-h.p. gasoline engine. At Springhill Station same type equipment was installed, with daily capacity of 7,500,000 gallons. A new building contains dual drive centrifugal booster pump with 50-h.p. induction motor and 60-h.p. gasoline engine. All pumping equipment is designed and installed for either electric or gasoline motor drive. All pumps are equipped with Cochrane flow meters of indicating, recording, and integrating type. Source of supply is unfiltered spring water. Synchronous motors are wound for 2200 volts and are of line starting design. Booster pumps are automatically controlled by float switches. All pumping units are driven through flexible couplings of DeLaval type, so that either gasoline or motor can be uncoupled on short notice. The old steam units have been held in reserve, giving each station three kinds of power.—*Lewis V. Carpenter.*

**When a City is Liable for Injuries Caused by Water Works Vehicles.** LEO T. PARKER. Water Works Eng., 84: 12, 883, June 17, 1931. Since operation of water department by municipality is ministerial function, city is liable for negligence of drivers to same degree as are private water companies and individuals. Court decisions held that if driver is without the scope of his employment, municipality is not liable. Cites case where driver should have gone direct to garage, but went out of his way, hauling another employee, and municipality was not held liable for damages. In several other cases, in which driver was off his route a few blocks, employee was held liable. Matter of scope of employement is rather a fine point. Both pedestrians and motor vehicles have equal rights on highways. If driver can prove that he used care to prevent striking a pedestrian, damages cannot be collected. But city must employ competent drivers for their trucks. It is legal duty of city to see that driver is mentally normal and experienced in driving motor vehicles. An act of negligence is a lack of ordinary care. Drivers should be informed of their liability and given specific instructions as to their duties. If driver kills someone unlawfully, he can be prosecuted for involuntary manslaughter.—*Lewis V. Carpenter.*

**Old Concrete Pump Pit Transformed into Modern Water Plant.** HOWARD K. BELL. Water Works Eng., 84: 12, 880, June 17, 1931. This pit, 36 feet in diameter and 30 feet deep, formerly housed two pumps geared to natural gas engines and used to supply water from Big Sandy River to Louisa, Kentucky, which has population of 2500. Plant now located in it includes circular settling basin, with central downcomer, into which water enters from riser pipe in center of downcomer after passing through mechanical mixing chamber, Settled water passes to filters. Introduction of meters has so reduced con-

sumption that plant operates only about 3 hours a day, giving 24 hours settling. Basin is of type used in some water softening plants. It has a flat main bottom over which is a conical false bottom sloping at about 45° from walls to center, where central sludge valve can be used to flush out greater part of the sediment without emptying basin. A natural-gas-engine-driven electric generator furnishes power for electrically-driven centrifugal pumps. An old 300-g.p.m. pressure filter unit was remodeled as wash water tank. This underground structure has all the advantages of a new modern filter plant at greatly reduced cost.—*Lewis V. Carpenter.*

**Experiences With Taste and Odor Control at Filter Plant.** C. F. NORMAN Water Works Eng., **84**: 3, 962, July 1, 1931. Mamaroneck, N. Y., tried a variety of treatments for removing tastes from water supply. Fish and eels smashed by pumps caused cod liver oil taste which was treated with both permanganate of potash and super-chlorination. Two parts of chlorine to one part of ammonia was used for algae. To stop growth of algae in filter gravel, chlorine was fed into wash water to give red residual by tolidine test. When this point was reached, chlorine was shut off and filter washed until faint yellow reaction was obtained. Copper sulphate is fed into water with the alum at rate of 3 pounds per m.g. to stop algae growth in coagulation basin. This process materially increases filter runs. Mud balls were eliminated by addition of flake caustic soda to cover sand, using about 300 pounds for a 1¼ m.g. filter. Twelve hours later, air-wash is applied, which thoroughly mixes sand and solution. Liquid is later drawn off and any mud balls remaining are broken up by wire mesh scoop. Person applying the solution should wear rubber boots and gloves and use gas mask.—*Lewis V. Carpenter.*

**Aërogenes Differentiation in Water Analysis.** C. C. RUCHHOFT, J. G. KALLAS, BEN CHINN and E. W. COULTER. Jour. Bact. **21**: 407–41 1931. See THIS JOURNAL, 23: 4, 565–581, 1931. An effort to obtain additional information in this study. It indicates that isolation of pure cultures by present standard methods is not trustworthy. Mixed cultures are frequently found on eosin methylene blue plates, leading to improper interpretation of final test when checked by GRAM stained smears. Criticism expressed seems fair and well founded. Further investigation is required, in order to discover a suitable medium.—*Edward S. Hopkins* (*Courtesy Chem. Abst.*).

**Interpretation of Water Analysis.** D. S. MCKINNEY. Ind. Eng. Chem., Analyt. Ed., **3**: 192–7, 1931. Two statistical methods for calculating concentration of anions of carbonic acid and of ortho-phosphoric acid are given. Factors underlying this method appear to be based upon certain physical-chemical theory. Relationship between dissociation constants of these substances and pH value of the water is shown. Excellent tables are given which reduce the complex calculations to simple arithmetic. The figures were obtained for systems at 25°C., the corresponding relationships at boiler temperatures have yet to be established. The information is of practical value in boiler feed water studies.—*Edward S. Hopkins* (*Courtesy Chem. Abst.*).

**Determination of Carbon in Sewage and Industrial Wastes.** F. W. MOHLMAN and G. P. EDWARDS. Ind. Eng. Chem., Analyt. Ed., **3**: 119–23, 1931. New modified chromate method for determination of carbon, using special apparatus, is given. Complete description of procedure and diagram of bulb required are included. Object of method is to aid in evaluating relation between carbon and the first stage of bio-chemical oxidation. [It is believed to be adaptable to water work for consumed oxygen testing—ABST.]—*Edward S. Hopkins (Courtesy Chem. Abst.).*

**Separation of Calcium and Magnesium by Molybdate Method.** R. C. WILEY. Ind. Eng. Chem., Analyt. Ed., **3**: 127–9, 1331. Method depends upon insolubility of calcium molybdate and solubility of magnesium molybdate in water. Precipitation can be made from slightly ammoniacal, or neutral, solution left after usual iron determination. Results, as shown by tables, are in close agreement with theoretical figures. While developed for limestone analysis, this method would appear to be applicable also to water work.—*Edward S. Hopkins (Courtesy Chem. Abst.).*

**Products of Corrosion of Steel. Factors Determining Their Composition and Its Influence on Rate of Corrosion in Oxygenated Water.** H. O. FORREST, B. E. ROETHELI and R. H. BROWN. Ind. Eng. Chem., **23**: 6, 650–3, June, 1931. Gelatinous $Fe(OH)_3$ and granular $Fe_3O_4$ exert major influences on corrosion rates of steel, or iron, in water. $Fe(OH)_2$ is formed in unbuffered water only above pH 8.0. At lower pH values iron present is converted, upon oxidation, to $Fe(OH)_3$, with subsequent precipitation. Reactions between $Fe(OH)_2$ and $Fe(OH)_3$ produce black granular $Fe_3O_4$, which is not resistant to diffusion either of oxygen, or of ions. Gelatinous $Fe(OH)_3$ retards diffusion, thereby preventing high concentrations of OH-ions at metal surfaces and promoting formation of corrosion-resistant film.—*Edw. S. Hopkins (Courtesy Chem. Abst.).*

**Recent Developments in Corrosion Prevention of Ferrous Metals.** V. V. KENDALL & F. N. SPELLER. Ind. Eng. Chem., **23**: 735–43, 1931. Review of work of past few years.—*Edw. S. Hopkins (Courtesy Chem. Abst.).*

**Prevention of Silicate Scale with Sodium Aluminate.** C. H. CHRISTMAN, J. A. HOLMES, and H. THOMPSON. Ind. Eng. Chem., **23**: 6, 637–46, June 1931. Salts of calcium and magnesium are usually regarded as principal boiler-scale forming agents. This study shows prevalence and serious effects of silicate scale. Use of sodium aluminate either as external, or as internal, boiler coagulant precipitates calcium, or magnesium, alumino-silicate; compounds which contain silicate as well as other incrustants. Precipitate is flocculent and can be easily removed. Advantages of aid of sodium aluminate in hardness removal are well known. Comprehensive data from results obtained with experimental boiler operating at high pressure are given.—*Edw. S. Hopkins (Courtesy Chem. Abst.).*

**Iodine Deficiency and Goiter.** C. A. HELLWIG. Archives of Pathology, 11: 709. Abstract from Jour. Amer. Med. Assoc., 97: 5, 353, August 1, 1931. Attempts by HELLWIG to produce goiter in white rats by feeding a diet poor in iodine gave only negative results. Excess of calcium in the drinking water and low intake of iodine caused marked epithelial hyperplasia of the thyroid. Drinking water rich in calcium and a diet rich in iodine produced small colloid goiters. Insufficiency of iodine was shown not to be the essential cause of goiter.—*J. H. O'Neill.*

**The Analysis of Boiler Scales.** NORMAN S. MOTT. The Chemist Analyst, 20: 4, 4–5, 1931. An outline method for determining $SiO_2$, $R_2O_3$, Ca, Mg, sulphate sulphur, sulphide sulphur, phosphorus pentoxide, ignition and blast loss, bulb water, and water soluble. Calculations and interpretation are shown.—*G. L. Kelso.*

**A New Test for Fluorides.** B. KOONE. The Chemist Analyst, 20: 4, 14, 1931. Reagent used is colloidal aqueous suspension of red lake formed when zirconium salt is mixed with alizarin. Dissolve 0.5 gram alizarin, 1, 2-dihydroxyanthraquinone, in 200 ml. alcohol: dissolve 1.5 grams zirconium chloride in 75 ml. alcohol. Solutions are mixed and precipitate formed is collected and dissolved in 25 ml. alcohol. Five ml. of this solution added to 100 ml. water constitutes test reagent, 0.5 ml. of which is added to 2.5 ml. of material to be tested which has been made acid with equal volume of concentrated HCl. In presence of more than 0.3 mg. fluoride, color change is immediate. Change comes within 15 seconds in concentrations of fluoride between 0.03 and 0.3 mg. Sulfates, oxalates, and phosphates give misleading color after 15 seconds. Interference from halogens can be avoided by addition of $Na_2SO_3$ before acidification. Complex fluoride ions give positive test.—*G. L. Kelso.*

**Cadmium as Reducing Agent for Trivalent Iron.** JAMES BRINN. The Chemist Analyst, 20: 4, 7, 1931. Volumetric determination of iron with $KMnO_4$ requires reduction to bivalent state. Reduction with lead, or zinc, is objectionable because filtration is required. Cadmium is furnished in convenient form for this use.—*G. L. Kelso.*

**Label Lacquer.** A. H. TAYLOR. The Chemist Analyst, 20: 4, 10, 1931. Pyroxylin handles from tooth brushes dissolved in butyl acetate, acetone, or ether alcohol, give a solution useful as label lacquer, as glue, and as glass cement.—*G. L. Kelso.*

## NEW BOOKS

**The Protection and Decoration of Concrete.** MAXIMILIAN TOCH. New York: D. Van Nostrand Co. Cloth; pp. 49. $2. Reviewed in Eng. News-Rec., 106: 369, February 26, 1931.—*R. E. Thompson.*

**Hydraulic Diagrams for Channels in Earth.** R. G. KENNEDY. 2nd. edition. Reprinted for sale by M. C. (Mrs. R. G.) KENNEDY, 182 Kew Road,

Richmond, Surrey, England. Flexible; 19 x 12 inches; 12 pp. of 4-column text, with 1 diagram; also 13 diagrams, lithographed in 3 colors. 17s. 6d. in England; postage extra. Reviewed in Eng. News-Rec., **106**: 370, February 26, 1931.—*R. E. Thompson.*

**The Technical Arts and Sciences of the Ancients.** ALFRED NEUBURGER. Translated by HENRY L. BROSE. New York: The Macmillan Co. Cloth; 7 x 10 inches; 518 pp. $10. Reviewed in Eng. News-Rec., **105**: 1018, December 25, 1930.—*R. E. Thompson.*

**Den Norske Ingeniorforenings Betonkomite. (First Report of Committee of Norwegian Society of Engineers on Action of Water on Concrete Dams).** Oslo: Norway. Den Norske Ingeniorforening. Paper; 7 x 10 inches; pp. 261. Reviewed in Eng. News-Rec., **105**: 1019, December 25, 1930.—*R. E. Thompson.*

**Piping Handbook.** J. H. WALKER and SABIN CROCKER. New York and London: McGraw-Hill Book Co., Inc. Flexible; 5 x 7 inches; pp. 763. $5. Reviewed in Eng. News-Rec., **105**: 1020, December 25, 1930, by CALEB MILLS SAVILLE.—*R. E. Thompson.*

**Silting of Reservoirs.** T. U. TAYLOR. Paper; 6 x 9 inches; pp. 170. Reviewed in Eng. News-Rec., **105**: 1020, December 25, 1930.—*R. E. Thompson.*

**Hydraulics.** FREDERICK CHARLES LEA. New York; Longmans, Green & Co.; London: Edward Arnold & Co. 5th. edition. Cloth; 6 x 9 inches; pp. 775. $7.50. Reviewed in Eng. News-Rec., **106**: 118, January 15, 1931, by JOHN H. GREGORY.—*R. E. Thompson.*

ERRATUM. Volume 23, October, 1931, page 1643, line 8. For "0.3 part per million" read "0.03 part per million."